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FARM IRRIGATION STRUCTURES

C. N. JOHNSTON¹

THE SUBJECT OF farm irrigation structures includes irrigation ditches or canals, irrigation piping systems, and the necessary controlling devices for applying the transmitted water. Since design and installation procedure for the canals have been carefully considered in hydraulics texts (1, 2, 3)² it need only be summarized here. Many controlling devices, especially those used in ditches, are not new, but have been developed in various districts of the West. As a result, colloquialism has often applied several names to the same device. Moreover, since a device may be viewed in different lights, the names were not always synonymous. For example, one structure used to hold back a portion of the flow in a ditch, keeping the water levels higher upstream, may be called a check by an observer on the upstream side. The same person, viewing the same structure downstream, might call it a drop because the downstream water levels were below those of the upstream. Because of this confusion, irrigation structures will be described here by their design details and not particularly by name.

The discussion considers these structures in the following order: (1) open channels such as canals or ditches; (2) ditch or canal structures and supplemental equipment; (3) pipe lines; (4) control devices in pipe lines.

Canals and ditches are the simplest and oldest means of transporting water in considerable volume. A discussion of the laws that govern flow in them will be useful later when considering flow in pipe lines. The second subject, ditch or canal structures and supplemental equipment, can logically be taken up after the ditches. Since these structures comprise a rather wide variety, examples of economical design in the range of structural materials will be presented so an operator can select a unit according to his needs. Pipe lines behave quite like ditches when water is being carried through them. Their points of superiority over the ditch, together with certain shortcomings, will be shown by comparison to aid the operator in deciding which carrier system to install. The last subject, control devices in pipe lines, covers the aids needed to make a pipe line work efficiently.

¹ Assistant Professor of Irrigation and Associate Irrigation Engineer in the Experiment Station.

² Italic numbers in parentheses refer to "References for Further Reading," at the end of this circular.

CANALS AND DITCHES

Canals and ditches are one method of transmitting water from source of supply to area of use. To transmit water they must have a slope or fall toward the destination. The greater the slope, the higher the velocity. If such channels are lined with nonscouring materials, they might be run down the side of a mountain. Under these conditions, the velocities would approach those of a free-falling body in the air. The flow speeds would be extremely unmanageable; and the water would swirl and tumble upon itself fiercely. In fact, ditch grades must be restricted so that the motion will be gentle and not swirling. Calm,

TABLE 1
MAXIMUM MEAN VELOCITIES SAFE AGAINST EROSION

Material	Mean velocity, in feet per second
Very light pure sand of quicksand character.....	0.75 to 1.00
Very light loose sand.....	1.00 to 1.50
Coarse sand or light sandy soil.....	1.50 to 2.00
Average sandy soil.....	2.00 to 2.50
Sandy loam.....	2.50 to 2.75
Average loam, alluvial soil, volcanic ash soil.....	2.75 to 3.00
Firm loam, clay loam.....	3.00 to 3.75
Stiff clay soil, ordinary gravel soil.....	4.00 to 5.00
Coarse gravel, cobbles, shingles.....	5.00 to 6.00

steady flow is more important in unlined ditches, where the swirling action cuts the bottom and sides (wetted surfaces), destroying the shape and eventually the grade. Even the lined channels, however, are cut by solids moving in the stream; and this cutting becomes more rapid as the velocities are increased. Primarily because destructive erosion does result from high velocity in canals and ditches, the grades are generally kept low (table 1). The quantity of water carried by a ditch is directly proportional to the average velocity. The necessity of keeping grades down demands, therefore, larger ditches than if higher velocities were permissible. Size, in turn, increases the cost and accentuates the advisability of accurate design.

The design of these channels is relatively simple, provided available data are utilized (1, 4, 5). Grade or slope, as already noted, influences velocity of flow. Another influence is smoothness: the water "drags" on a rough channel and glides more easily on a slick, smooth one. This slowing effect occurs only along the line of contact between water and channel face, or on the "wetted perimeter." A given volume, passing through a ditch at a fixed rate, will have a given depth. If the width of the ditch is doubled and the grade is kept the same as before and the condition of the lining is the same as in the original ditch, certain changes in the flow occur. Though the same amount of water is passing, the velocity and the depth of flow have changed. The depth is not half the former depth, nor is the velocity the same as before, though the slope

is the same. This change in velocities and depths results from the drag on the water caused by the wider ditch. In other words, the length of the surface (ditch width) of the conduit in contact with the water at any given cross section has been increased; and that change has altered the velocity and the depth. This surface of contact measured at a stream cross section is called the wetted perimeter and is given the mathematical symbol P . If, in this same widened ditch, sufficient water is admitted to double the depth of flow, the velocity will be about doubled. Here, again, is a new situation—a third possible velocity with constant slope.

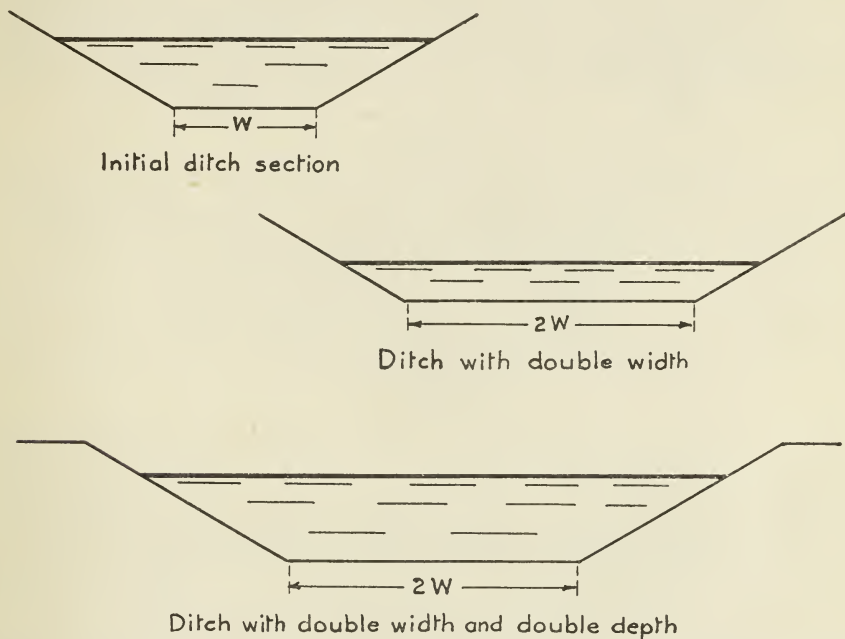


Fig. 1.—Ditch cross sections illustrating the effect of varying bottom width and depth.

Figure 1 illustrates the three conditions listed. The upper ditch in this figure is the initial ditch section, with the water-surface level shown. In the center detail, the ditch has been doubled in bottom width, or dimension w has become $2w$; and the same amount of water flowing requires a little more than half the depth of the initial ditch section. The ditch detailed in the lower third of figure 1 illustrates the same widened ditch (width equals $2w$) but with water twice as deep as in the one just above. As can be seen, doubling the depth of water in the widened ditch has not greatly increased the length of wetted cross section or wetted perimeter in the ditch. As a result, a much-increased cross-section area of water is moving over only a slightly increased wetted perimeter. In other words, the drag has not gone up much, but the area of flow has more than doubled, while the slope of the ditch bottom remained constant. Tests of ditches similar to those in figure 1 would demon-

strate that the average velocity in the ditch shown in the lower detail would be more than twice the velocity found for the ditch detailed in the drawing above it. The situation may be stated thus: the drag effect in the two ditches is similar, approximating equality; but the cross section of flow has so increased in the lower one that a greater body of water moves along in it at a distance from this surface of drag. Being farther away, this larger body of

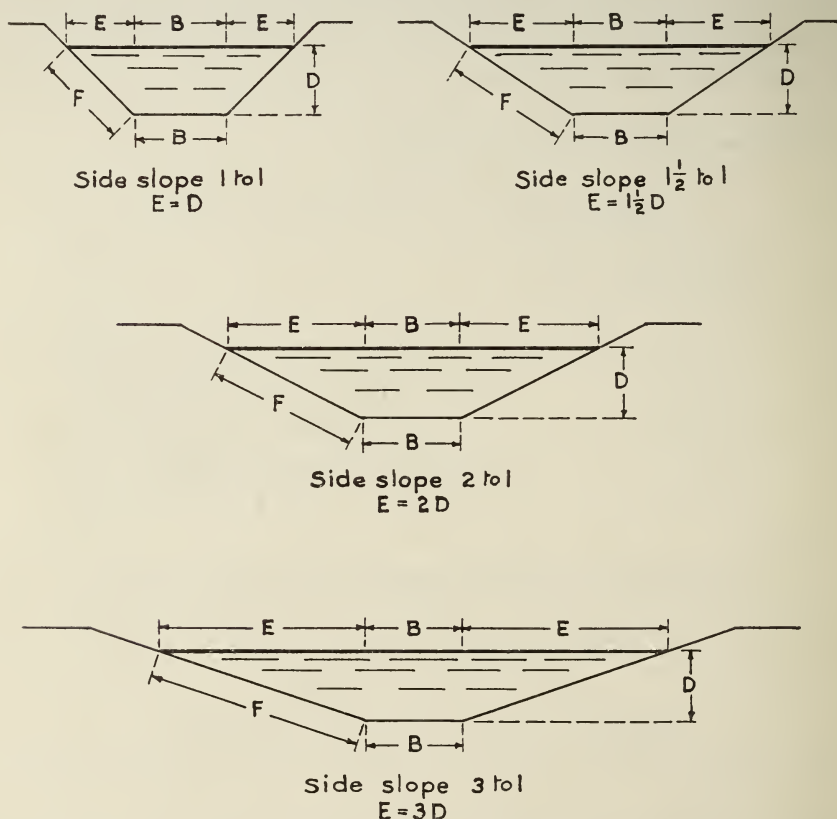


Fig. 2.—Typical ditch sections.

water moves more freely and faster than is possible where the depth of flow is shallow, as in the middle detail of the figure. Such are the conditions in actual ditches. In studies made on ditch flow, such investigators as Ganguillet and Kutter (6) have found that the cross-section area (represented by the symbol A) of the stream divided by the wetted perimeter, P , gives a value, R , that can be used to interpret the average value of velocity from the slope for any given channel. The formula would be $\frac{A}{P} = R$. R is called the hydraulic radius, and is a mathematical term applied to flow in all channels and pipes. When the average or mean velocity is known, the volume of water moving $= A \times V$; or area in square feet, times velocity in feet per second, equals the

quantity in cubic feet per second—sometimes represented as Q . The drag of the wetted surface is another variable. Every obstruction (brush, grass, roots, stones, sand or gravel on the bottom or in the lining material of a channel) resists the motion of the flowing stream. Even the smoothest surfaces create some drag through adhesion of the water itself. Any obstruction tends to reduce the velocity along the surface of contact. The term used to identify the obstructing effect of the surface is its “roughness,” symbolized for mathematical calculations by the letter n . The value n becomes part of an equation that expresses the flow relation for any given ditch.

Unfortunately, most ditches and canals are not simple square-cornered channels whose cross-section area is the product of width and depth. As a rule they have sloped sides and a flat bottom (fig. 2). Both bottom width and

TABLE 2
DITCH CROSS-SECTION RELATIONS DEVELOPED FROM FIGURE 2

Side slope ratio, E to D	Value E	Length F	Wetted perimeter, $P = F + B + F$	A , Area, in square feet
1 to 1.....	D	$D\sqrt{2} = 1.414 \times D$	$B + 2.828 \times D$	$(B + D) \times D$
$1\frac{1}{2}$ to 1.....	$1\frac{1}{2}D$	$D\sqrt{3.25} = 1.803 \times D$	$B + 3.606 \times D$	$(B + 1\frac{1}{2}D) \times D$
2 to 1.....	$2D$	$D\sqrt{5} = 2.236 \times D$	$B + 4.472 \times D$	$(B + 2D) \times D$
3 to 1.....	$3D$	$D\sqrt{10} = 3.162 \times D$	$B + 6.324 \times D$	$(B + 3D) \times D$

slope of sides vary in different ditches. The solutions, however, for cross-section area and wetted perimeter are relatively simple even for these somewhat complex shapes. Lined ditches and canals can have side slopes as great as 45 degrees (1 to 1) or slightly steeper. Unlined earthen channels cannot retain their shape if steep sides are constructed: the wetted sides will slump by their own weight till the bank becomes stable. This slumping fills the channel so that the flow capacity decreases. To eliminate this deterioration, the constructed banks are given the flatter slopes initially. Clays can stand up with $1\frac{1}{2}$ to 1 slopes (fig. 2); but sandy materials require as much as 3 to 1 slopes if traversed by ditches, where dimension E (fig. 2) is three times dimension D . Other soil types require ditch bank slopes between these extremes.

Regardless of the slope used in the ditch sides, the cross-section area of the stream in square feet is approximately equal to half the width of the bottom plus half the width of the stream surface multiplied by the depth. In the upper left detail of figure 2, dimension E equals dimension D , because the slope is 1 to 1; so the surface of the stream $= E + B + E = D + B + D = B + 2D$. Now if the width of the bottom is added to the width of the surface, the result is $B + 2D + B = 2B + 2D$. If these terms are divided by 2, the result is $\frac{2B + 2D}{2} = B + D$, the average width. Multiply this by the depth to get the

area, $A = (B + D)D$. If the slope were $1\frac{1}{2}$ to 1 instead of 1 to 1, as in figure 2 (upper right hand), then E would equal $1\frac{1}{2}D$ and the area A would

be $(B + 1\frac{1}{2}D)D$ by the same reasoning as above. Similarly, if the side slopes were 3 to 1, then $E = 3D$ and $A = (B + 3D)D$ (see table 2).

The wetted perimeter is the bottom width, B , plus the two sides F in figure 2, of the stream. Since the length of the sloping sides is a number involving the sums of squares of numbers and their square roots, the easiest way to handle them is to use a tabulation showing the values as in table 2.

As a typical application, assume a bottom width of 3 feet, a side slope of 2 to 1, and a depth of 2 feet. From line 3 of table 2, the wetted perimeter for a 2-to-1 side slope $= B + 4.472 \times D = 3 + (4.472 \times 2) = 3 + 8.944 = 11.944 = P$. Area for this assumed condition is $A = (B + 2D)D = (3 + 2 \times 2) \times 2 = 7 \times 2 = 14$ square feet. The hydraulic radius, $R = \frac{A}{P} = \frac{14}{11.944} = 1.172$ feet.

One cubic foot per second is equivalent to the following volumes:

- 1 cubic foot per second = 1,728 cubic inches per second
- = 60 cubic feet per minute
- = 3,600 cubic feet per hour
- = 448.83 gallons per minute (approximately 450 gallons)
- = 50 southern California miners' inches
- = 40 California statute miners' inches
- = 0.992 acre-inch per hour (approximately 1 acre-inch)
- = 1.984 acre-feet per day (approximately 2 acre-feet)

With these data available, a series of charts could be drawn to express each equality. It is desirable for users of water to know the units. One of the commonest conversions, however, is from gallons per minute (g.p.m.) to cubic feet per second (c.f.s.). If 1 c.f.s. = 448.83 g.p.m., then 2 c.f.s. = 897.66 g.p.m., and so forth. Figure 3 portrays this relation. The dotted lines indicate the procedure in using the chart. Starting at the scale for gallons per minute, on the left, move horizontally to the sloped line and from that intersection, vertically, down to the bottom of the chart, reading the cubic feet per second at that intersection. The chart operates as well if the cubic feet per second are known and the gallons per minute are to be determined; the procedure in that case is reversed. For general checking, the value 1 c.f.s. = 450 g.p.m. is close enough.

Included, for convenience, are three graphs (figs. 4, 5, 6) from which one can obtain the expected velocity of flow for any grade and for the three roughness values, $n = 0.015$, 0.025 , and 0.035 . These graphs, drawn from the proved data (13) of many field and laboratory experiments, are suitable for most farm-ditch designs if a safe value of n is used. Values of n may range from 0.010 to 0.060. A safe value is larger than will probably be found; the three values given in the chart are safe for use with the ditch type described for each.

A ditch must slope toward the point of delivery. This term, obvious in meaning, is used synonymously with *fall* or *grade* to describe the change of elevation of the ditch bottom or the stream surface for a given distance. For example, the slope of a ditch 3,000 feet long may be 0.5 foot per 1,000 feet.

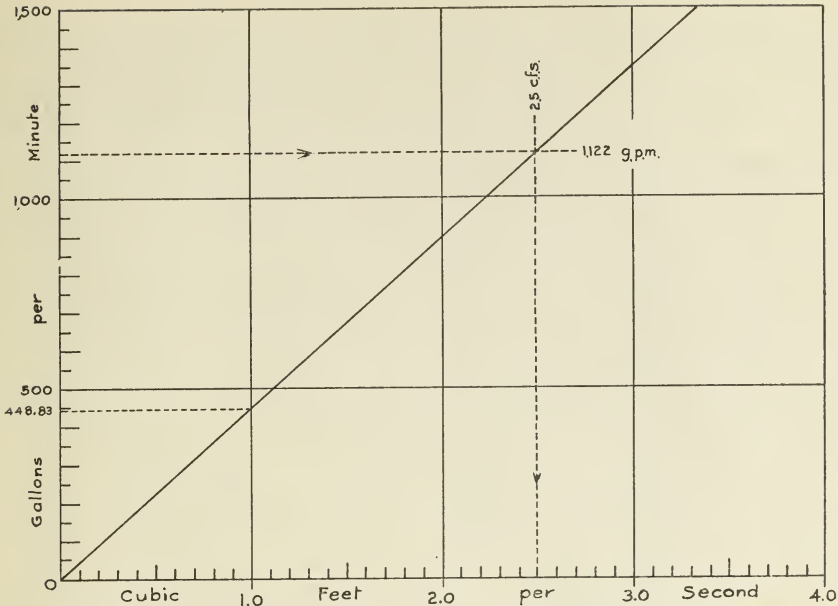


Fig. 3.—Conversion chart—gallons per minute to cubic feet per second.
For explanation see text, page 6.

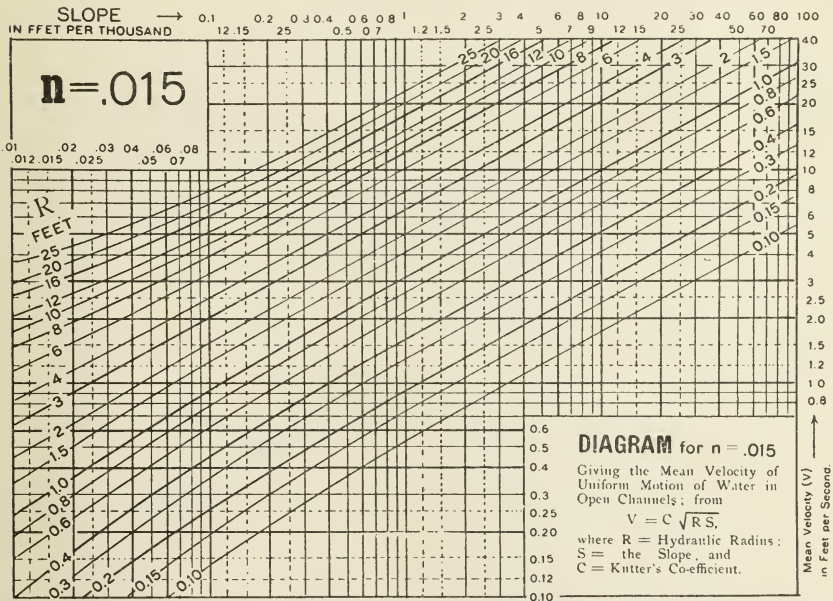


Fig. 4.—Value of n , 0.015, for ditches having very smooth beds—for example, wood-lined channels using unplanned boards, not in perfect order because of inferior workmanship or age; slightly roughened concrete; ordinary brickwork; smooth stonework; foul and slightly corroded iron. These conditions might be called very good. (Diagram reproduced from reading-reference 13, based on Kutter's formula.)

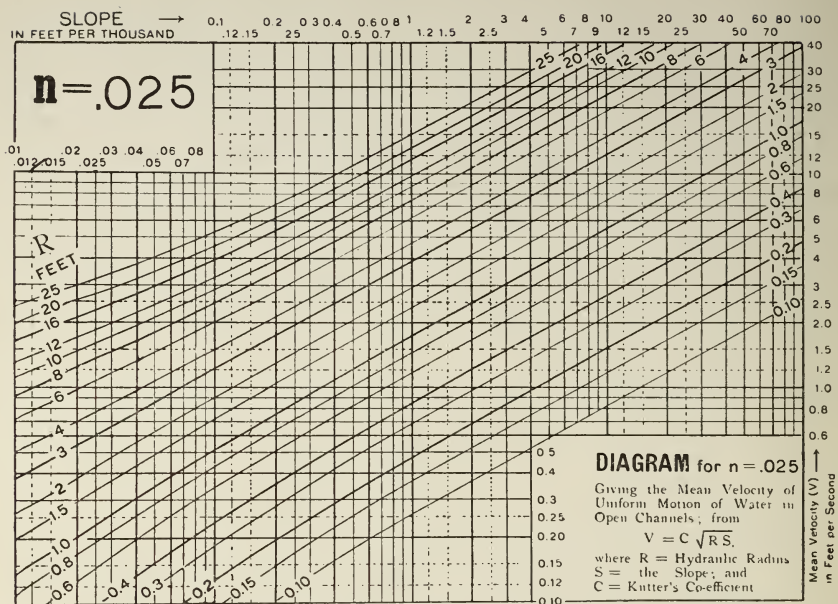


Fig. 5.—Value of n , 0.025, for average ditches, reasonably clean but with fairly rough beds—for example, fairly new well-formed channels lined with natural sediment or composed of loose coarse gravel. These conditions are about average. (Diagram reproduced from reading-reference 13, based on Kutter's formula.)

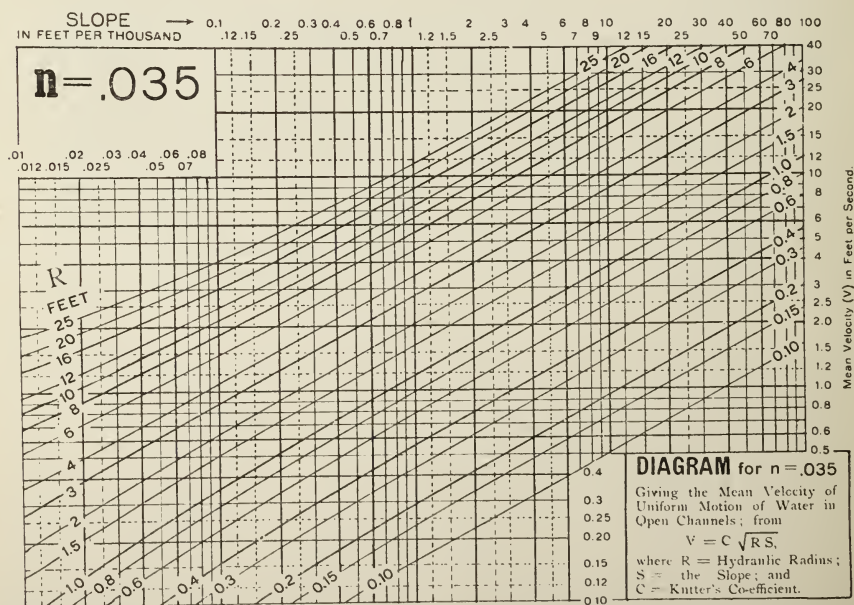


Fig. 6.—Value of n , 0.035, for weed-filled ditches with rough beds—for example, channels in bad order, with stones and trash strewn on the bottom and sides, or about one-third full of vegetation. These are very poor conditions. (Diagram reproduced from reading-reference 13, based on Kutter's formula.)

In that case, the fall in the complete ditch is 1.5 feet; and the fall per 1,000 feet is 0.5 foot. The grade is 0.5 foot per 1,000 feet or 0.05 foot per 100 feet or 0.0005 foot per 1 foot. The fall, grade, or slope of a ditch may also be expressed as so many feet per mile. For the ditch discussed above, the fall would be $0.5 \times 5.280 = 2.64$ feet per mile. Sometimes when the unit distance is understood—for example, per 1,000 feet or per mile—the slope, grade, or fall is said to be simply 2.64 feet or 0.5 foot, which, for 2.64 feet per mile, would be equivalent to 0.5 foot per 1,000 feet. The following discussion will identify the unit of length in discussing these three synonymous terms. Another term, *friction loss*, is used interchangeably with *fall*, *drop*, or *grade*. Friction loss results largely from the drag or roughness characteristics of the wetted surface of the conduit; it equals the fall or drop in the water surface between any two

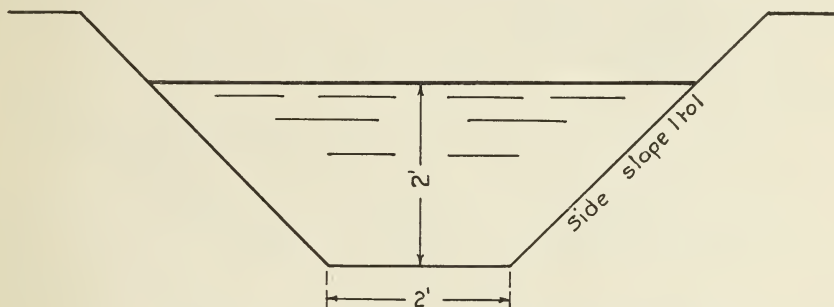


Fig. 7.—Ditch section with depth, bottom width, and side slope known.
See problem 1, page 9.

points in the stream. To illustrate the use of these ditch-velocity charts (figs. 4, 5, 6), several typical problems are given below:

Problem 1.—A concrete-lined canal (fig. 7) in fair condition has a slope of 0.4 foot per 1,000 feet. What is its velocity and capacity? For this type of canal, a safe value of $n = 0.015$. The cross-section area,

$$A = \frac{\text{water surface width} + \text{bottom width}}{2} \times \text{depth (from table 2)}$$

$$= \frac{(2 + 4) + 2}{2} \times 2 = 8 \text{ square feet.}$$

$$\text{The hydraulic radius} = \frac{A}{P} = \frac{A}{B + (2.828 \times D)} \quad (\text{from table 2})$$

$$= \frac{A}{P} = \frac{8}{P} = \frac{8}{2 + (1.414 \times 2)} = \frac{8}{7.656} = 1.045 = R.$$

From the chart on figure 4, with a hydraulic radius of 1.045 and a slope of 0.4 foot per 1,000 feet, the mean or average velocity should be $2.0 \pm$ feet per second.

With a cross-section area of 8 square feet and a velocity of 2.0 feet per second, the quantity of water in motion would be $8 \times 2.0 = 16.0$ cubic feet per second.

Problem 2.—Reverse problem 1: determine what depth of water will flow in a 2-foot wide ditch bottom with 1 to 1 sides having concrete lining in fair condition, and with a slope of 0.4 foot per 1,000 feet when 16.0 cubic feet is being carried. Roughness, n , = 0.015. Assume that water 3 feet deep is required, and that problem 1 is not available.

$$A = (B + D)D = (2 + 3) \times 3 = 15 \text{ sq. ft. (from table 2)}$$

$$P = B + (2.828 \times D) = 2 + (2.828 \times 3) = 10.484 \text{ sq. ft. (from table 2)}$$

$$R = \frac{A}{P} = \frac{15.0}{10.484} = 1.43 \text{ ft.}$$

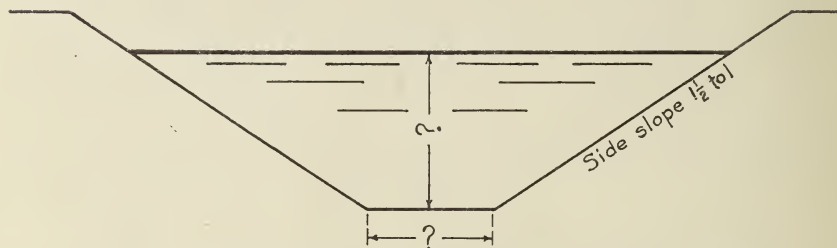


Fig. 8.—Ditch section with depth and bottom width unknown, but with side slope known. See problem 3, page 10.

We must now check to see whether correct depth has been assumed. For $R = 1.43$, slope $S = 0.4$, and the coefficient $n = 0.015$ for concrete lining of this condition, we find from the chart in figure 4 that the average velocity = 2.5 feet per second and the quantity handled = $2.5 \times A$ (area) = $2.5 \times 15.0 = 37.5$ cubic feet per second, which is too much. Evidently, too great a depth has been assumed: both the area and the velocity are excessive, judging by the solution to problem 1. Lacking problem 1 to go by, as would be the case in normal design, another and smaller assumed depth would be taken, and another computation would be made. A third or fourth computation might be needed to determine the exact depth, which would prove to be 2 feet as in problem 1.

Problem 3.—What size unlined clean earth-bank ditch would be necessary to carry 2 c.f.s. with a slope of 0.2 foot per 1,000 feet? The value of n for friction coefficient is 0.025 for a clean earth-bank ditch. The slopes of a dirt-side ditch should be less steep than in a lined one because if the slope is too great, the earth will slide in on becoming wet. A slope of 1.5 to 1 is satisfactory and can be assumed (fig. 8). Assume a bottom width of 1 foot and a depth of 1.6 feet.

$$A = (B + 1\frac{1}{2}D)D = (1 + 2.4) \times 1.6 = 5.44 \text{ square feet (from table 2)}$$

$$P = B + 3.606D = 1 + (3.606 \times 1.6) = 6.77 \text{ (from table 2)}$$

$$R = \frac{A}{P} = \frac{5.44}{6.77} = 0.804.$$

For $R = 0.804$, slope $S = 0.2$ foot per 1,000 feet, and $n = 0.025$, velocity = 0.65 (see chart, figure 5). The flow = $A \times V = 5.44 \times 0.65 = 3.54$ c.f.s., which is too large. In design, one can stop here, because evidently a ditch with the assumed dimensions will have over one and one-half times the desired capacity—an adequate safety factor. To get closer to the true flow depth, we might make another calculation. For a depth of 1.3 feet,

$$A = (B + 1\frac{1}{2}D)D = (1 + 1.95) \times 1.3 = 3.835, \text{ or } 3.84 \text{ (from table 2)}$$

$$P = B + 3.606D = 1 + (3.606 \times 1\frac{1}{2}) = 1 + 4.69 = 5.686 \text{ or } 5.69$$

(from table 2)

$$R = \frac{A}{P} = \frac{3.84}{5.69} = 0.67.$$

Under these assumptions, then, velocity $V = 0.58$ (from chart, fig. 5). The flow = $3.84 \times 0.58 = 2.22$, which is about 11 per cent over capacity and close enough on the safe side.

Problem 4.—Suppose we desire to run 1,350 g.p.m. a distance of 800 feet between two points having elevations of 452.64 and 452.40 feet, respectively. Since the water supply is contaminated with weed and grass seed, the earth-bank ditch will be quickly overgrown, and the flow impeded. How large a ditch is necessary? We know that 1,350 g.p.m. = 3 c.f.s. (fig. 3). The value of friction coefficient n for these weedy ditch conditions = 0.035. (Assume worst conditions, which will develop soon in this ditch.) The slope or grade of ditch = 452.64 feet — 452.40 feet = 0.24 foot per 800 feet or 0.3 foot per 1,000 feet. Assume a $1\frac{1}{2}$ -foot-wide bottom to ditch, $1\frac{1}{2}$ -to-1 side slopes, and a depth of flow of 1.5 feet. (This guess is made from the solution to problem 3, where a ditch with a 1-foot-wide bottom, flowing 1.3 feet deep, carried 2.22 c.f.s.)

$$A = (B + 1\frac{1}{2}D)D = (1.5 + 2.25) \times 1.5 = 5.62 \text{ sq. ft. (from table 2)}$$

$$P = B + 3.606D = 1.5 + (3.606 \times 1.5) = 1.5 + 5.41 = 6.91$$

$$R = \frac{A}{P} = \frac{5.62}{6.91} = 0.81.$$

For $n = 0.035$, $R = 0.81$, and slope 0.3 foot per 1,000 feet, the average velocity should be 0.55 foot per second (from chart, fig. 6).

The quantity would be then $0.55 \times 5.62 = 3.1$ c.f.s.; and the depth, 1.5 feet, would appear to have been a very lucky guess, together with the assump-

tion that the bottom width is 1.5 feet. Obviously, no ditch should be made just deep enough to handle the computed depth of water. Additional depth not only gives a safety factor for settlement, but adds bank height above the water level to prevent spilling over or erosion at the sides. Stock or other trampling will then not be immediately destructive.

These examples and their solutions show the typical calculations necessary in designing ditches. By using the three charts covering friction coefficients and the simple formulas given above, one can solve many water-distribution problems on the farm. For a large job involving many ditches and considerable expense, one should obtain some hydraulics text (1, 4, 5) to supplement the

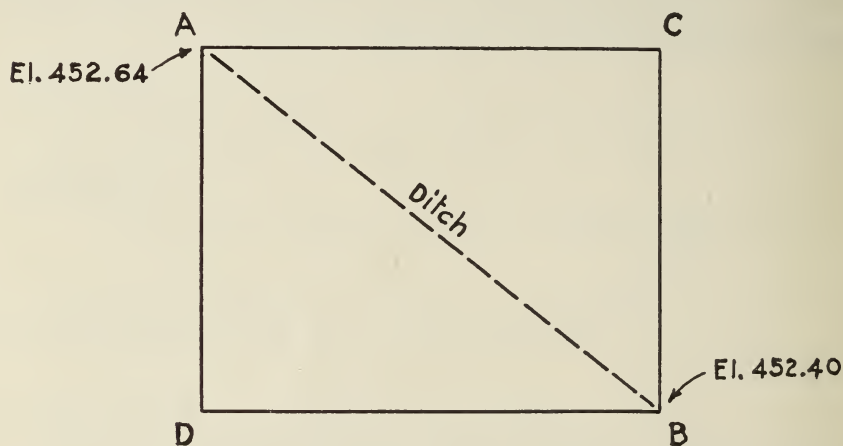


Fig. 9.—Typical field with ditch diagonally across it showing elevations in feet.

material available herewith or obtain the services of an engineer. Data available in such publications include a wide range of values for n , the friction coefficient; a truly economical design can be completed by their use. This does not mean that the three values for n (0.015, 0.025, and 0.035) are not subject to economic application to field conditions. They provide a rough check of ditch sizes required for the three general conditions—lined, unlined, but clean, and unlined and weedy, or foul. Normally, such a rough check suffices for farm use: the unlined ditches, which are most common, are built approximately to dimension. For this reason the results of calculations must be on the safe side.

DITCH CONSTRUCTION

One of the foregoing design problems might have referred to the field shown in figure 9. Here water is to run from point A to point B , a distance (as in problem 4) of 800 feet with the water level at A , 452.64 feet, and at B , 452.4 feet. A ditch has been designed to carry 1,350 g.p.m. or 3 c.f.s. from A to B , utilizing the available slope and even allowing for the weedy conditions that will develop soon after the ditch flow starts. Construction is begun. Midway between A and B (fig. 10) is a 100-foot-long streak of clean gravel. Because,

obviously, such a streak will cause excessive seepage losses, the ditch must be lined in that zone. A new design problem for this 100-foot length of ditch arises and is solved as in the examples. It might be possible to go around the gravel strip, passing through *C* or *D* en route to *B*. Since, however, much additional distance would result and the slope would be reduced proportionally, requiring a larger ditch than in the original design, lining the 100-foot strip of the gravel zone might be the cheapest. Without checking the economics of the various routings, we may conclude that success depends upon careful preliminary studies, both by using surveying instruments and by inspecting the physical aspects of the proposed route. The ditch must be located on a path suitable from several aspects. First, it must be convenient for irrigation: that is, its water surface must be above the land. In general, therefore, it should follow a ridge or start from a high point. Second, it must have fall or drop in the direction of flow. If a uniform grade cannot be kept, we must

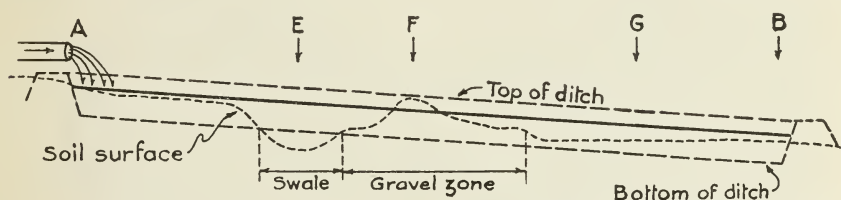


Fig. 10.—Profile of the ditch line across a field, shown in figure 9.

change the ditch dimensions by solving satisfactorily a problem for each grade used and for the flow desired. Third, the ditch should not interfere with cultivation operations any more than is necessary. This requirement may add additional length to a ditch. Figure 9 shows a possible bad location. If the ditch runs from *A* to *B*, the field is cut in two, forcing cultivation equipment to go a long way around unless bridges are supplied. Since the example, *A-B*, is a comparatively short ditch, 800 feet, the distances might not be too great if the equipment had to go around; but if *A* and *B* were much farther apart, bridges might be necessary. Each operator must decide how far convenience in operation should control the final layout.

Additional considerations can be drawn from ditch *A-B*, which will have both lined and unlined sections. An assumed profile along the path of *A-B* will illustrate many matters that must receive attention. In reality not all the procedures suggested would be warranted on such a short ditch; if all were used, they would complicate the construction unnecessarily. This statement will be clearer later on. The reader can then decide which method suits his conditions and can be followed out with available equipment. As a profile shows (fig. 10), the land surface is irregular. The ditch line is straight on the bottom and at the crest of the banks. The relative position of ditch section and present earth levels are shown as sections taken at points *A*, *E*, *F*, and *G* in figure 11. This figure will be used to illustrate the various construction procedures. Starting with section *A* at the left-hand end of the ditch, the sketch

indicates that more dirt will be cut to make the ditch than is necessary in fill to make the bank at this point. This condition is fortunate: any excess dirt can be used at section *E*, where a fill must be made to bring the grade up to the ditch line. If the ground surface were regular all the way from *A* to *B* and the

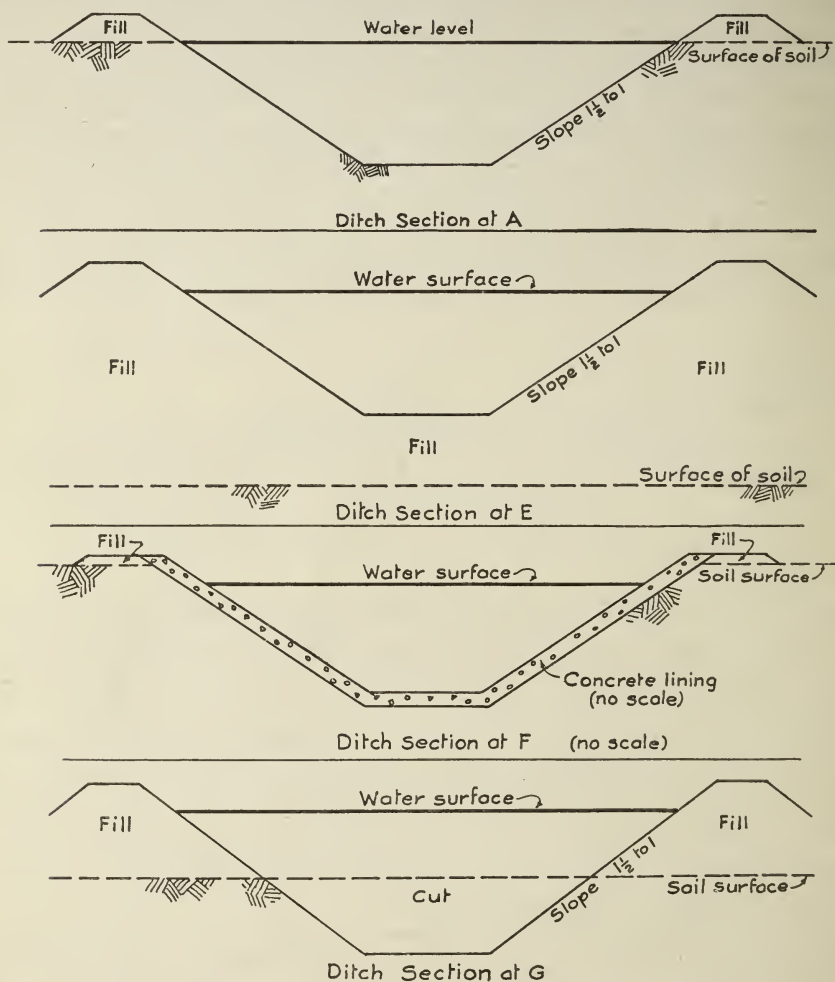


Fig. 11.—Details of cut, fill, and lining in the ditch across a field, showing construction details of locations designated in figure 10.

ditch were cut into the soil as at section *A*, the excess cut would become a problem. A solution might be to raise the bottom and the bank tops so the cut equalled the fill. Thus the excavated material placed on the adjoining uncut soil banks would make the fill necessary to form the ditch to the dimensions required. Cuts and fills balance up the cross-section area if the actual cut equals the cross-section area of settled fill or is slightly greater (not over 10 per cent). The material removed is loose and will settle; it should therefore be

piled higher on the banks than the final design requires. Unless such allowance is made for settlement, the banks may not be high enough to provide a safe freeboard. At the top they have a flat surface, whose width should not be less than 6 inches. This width gives protection against weathering and wave action. Tamping any fill will help consolidate it and reduce the hazard of leakage till settlement further consolidates it. One should not attempt to pack wet soil: the water has caused it to swell, and mechanical packing has little effect on this condition. Such soil shrinks as it dries, and the effort of packing is wasted because cracks appear and leakage is almost certain. Completely dry soil is equally poor packing material; a moist soil not wet enough to be plastic is

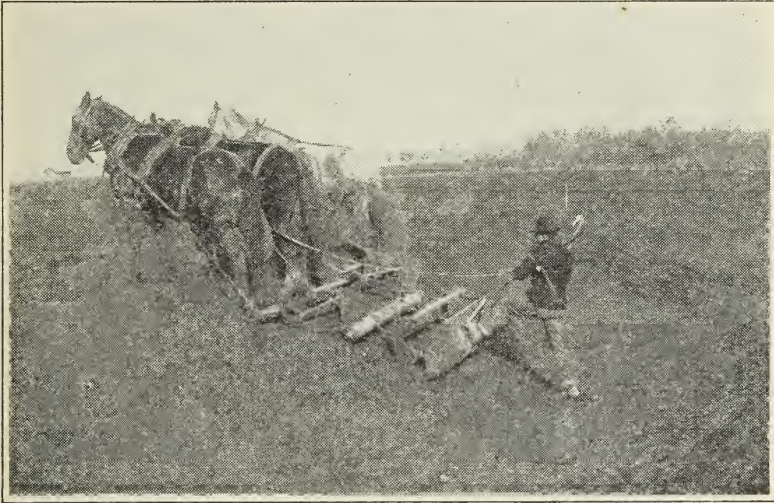


Fig. 12.—Fresno scraper in use making ditch banks.

best. Since the textures vary, no general rule is safe; one should experiment with each soil to determine the best condition for working. In general, a good condition prevails when plowing does not glaze the soil at the surfaces of contact with the plow. In the detail of section *E*, figure 11, the land surface is below the bottom of the completed ditch. Under these conditions the ditch must be built on a fill, which should be compacted as much as possible during construction operations. This section of the ditch at *E* resembles that for a ditch built by raising two parallel banks on the surface of the ground. The necessary dirt must be brought to the site from the adjoining surface—perhaps by a fresno scraper. Soil can be skimmed from the surface or off high points adjoining the ditch and dumped to form the two banks. In this procedure the teams or tractor, as well as the fresno scraper (fig. 12), cross and recross the accumulating fill, packing it down as work progresses. Final shaping is sometimes given the ditch by hand shoveling. In *E* (fig. 11), a rather large volume of material will be needed for fill. Some of this might come from site *F*, where a ridge rises above the water surface in the proposed ditch. Taking off this surface

soil would permit irrigation from the ditch at that point and would supply the soil deficiencies at *E* as well. If the material is used from *F*, the gravel zone, the gravel must be mixed and buried through the bottom of the fill, and watertight soil must make up the foot or so of thickness adjoining the ditch sides and

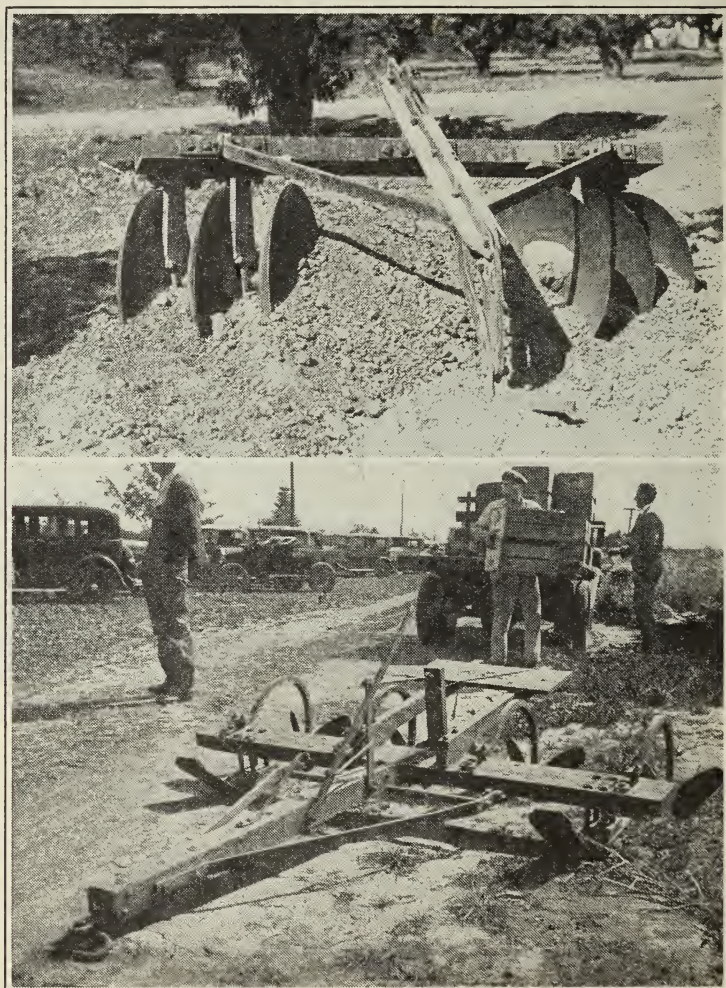


Fig. 13.—Upper view, disk ridger suitable for making a ditch bank. Lower view, four-blade furrowing implement for making several small ditches at once.

bottom. Such a fill must be packed as well as is feasible during its building. It should be settled by allowing water to stand in it as soon as convenient and before the water is permitted to flow through. After settlement, the filled area may need reshaping. A fill is a precarious location for a ditch and should be avoided if possible. Whenever the nearby soil is not suitable for holding water, clay should be brought in if possible, and a 6- or 8-inch layer of it should be

tamped in place in a filled section. Since the best soil on any farm lies in the first foot depth, avoid wholesale stripping of this layer to make fills.

Detail *F*, figure 11, shows the lined section in this ditch. The occasion for the lining is the gravel zone in figure 10 and in problem 4. A use for material removed at *F* has been suggested in the foregoing as part of the fill at *E*. If no large volume of fill is needed, the excavated material can be spread over a considerable area. In constructing the ditch at *F*, the normal procedure would be rough-shaping with disks or plowing and rough-shaping with the V-board shaper (figs. 13 and 14). Finish-trimming would be done carefully with shovels, to furnish a smooth surface on which the lining will be spread. The finished earth is cut 2 to 3 inches below the finished face of the lining and acts as the

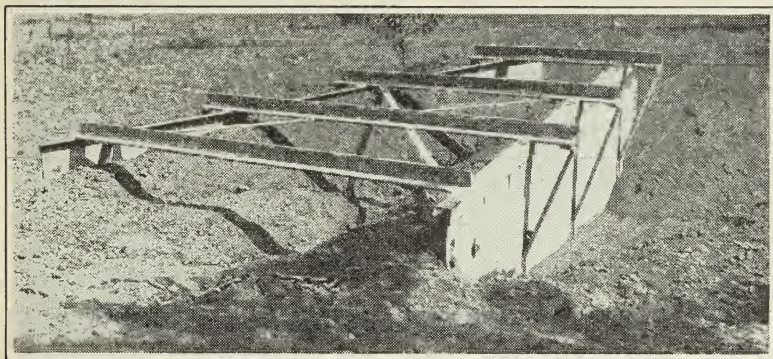


Fig. 14.—Metallic V-board ridger useful in forming ditch banks.

outside form for it (fig. 15). The lining, whether asphaltic or plain concrete, is troweled or bladed in place and smoothed as well as possible. Reinforcing may or may not be used. Smaller ditch linings are generally made without reinforcing. Expansion joints (fig. 16) are sometimes provided at 10- or 15-foot intervals on concrete linings. Three or 4 strips of tar paper in the joint between sections of concrete will act as expansion points. Concrete can be put on by means of a jet or “gunited” to the V-shaped ditch banks (fig. 17), forming a dense, satisfactory lining. This procedure is followed for large ditches, and steel reinforcing is generally used with it to bind the concrete. Troweling improves “Gunited” cement linings. Ditches are sometimes lined with concrete poured in place against forms.

Detail *G*, figure 11, discloses a situation somewhat like that in section *A*. There is, however, less cut than fill; some soil must be carried in to make the banks of the fill high enough and wide enough. The disks on the plow, together with the V-board shaper, can be used to cut the rough ditch. Then the necessary additional soil must be scraped in by a fresno. When that soil is in place and compacted, the final hand-trimming can be done.

All hand-trimming follows grades set by reference to surveyed stakes so the final grade is true. Frames of light wood, having the correct dimensions of the ditch, can be moved along during trimming to insure accurate slopes and level

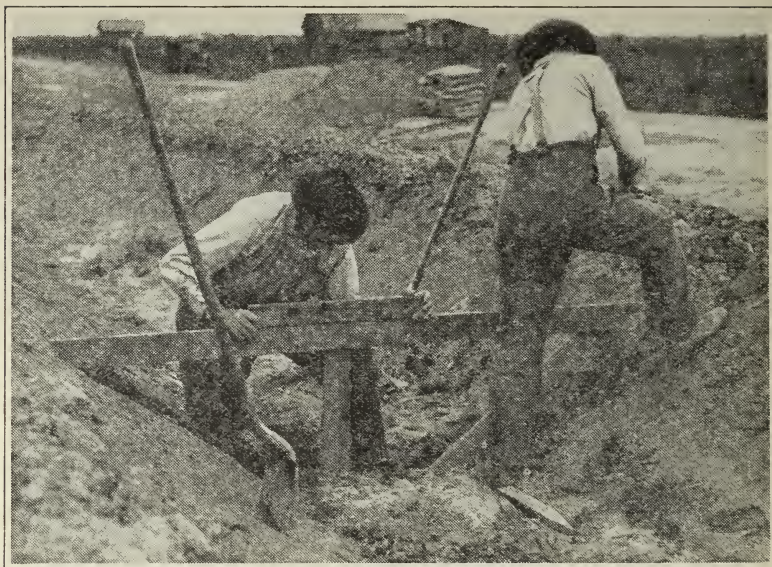


Fig. 15.—Hand-trimming a ditch to the dimensions of a template before pouring the concrete lining.

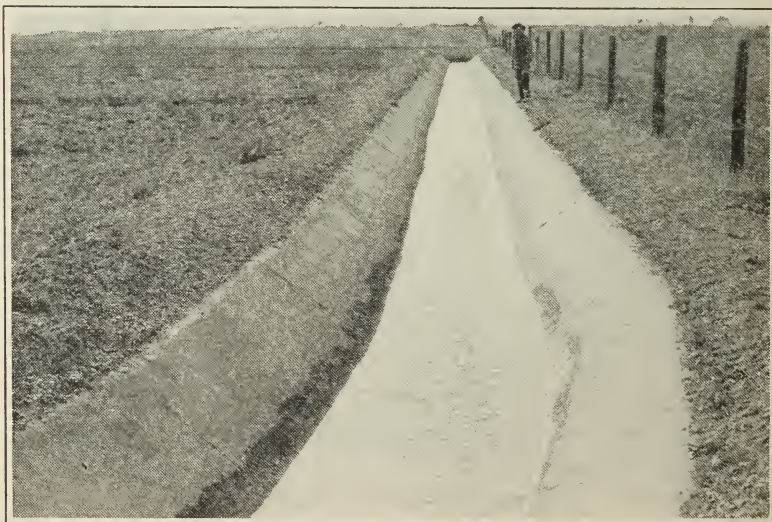


Fig. 16.—Concrete-lined ditch with expansion joints (showing as black joint lines).

bottoms. Figure 15 details such a frame, suitable for use in shaping the unlined sections of the ditch designed in problem 4. Many earthen-wall ditches are finished by the rough shaping processes alone. Such ditches cannot be as clean and true as hand-trimmed ones.

If the ditch A-B (fig. 9) must be bridged for a road crossing, a logical spot

would be at F (fig. 10), where the ditch is lined and lies entirely below the normal soil level. A typical bridge is detailed on page 36.

The filled section over which the ditch is carried at E (fig. 10) might be



Fig. 17.—Placing gunite cement on stucco wire reinforcing, with the trimmed earth serving as the outside form.

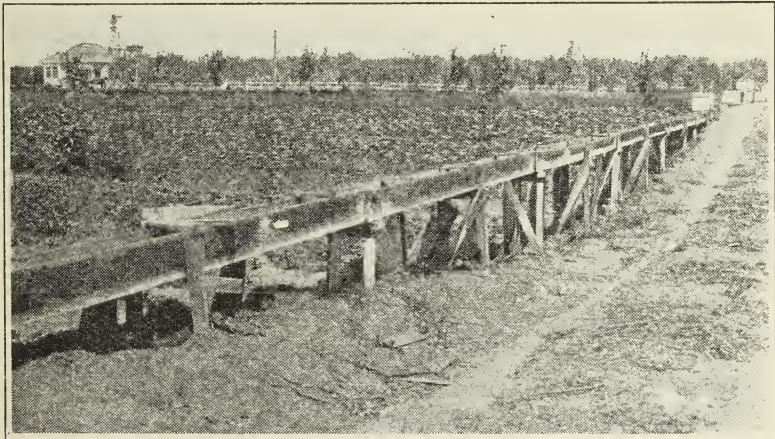


Fig. 18.—Typical small wooden flume across a depression.

replaced with a flume (fig. 18). This resembles a lined ditch of half-round, half-rectangular, or possibly triangular section made of wood or metal and resting on framing that maintains the grade line of the ditch. Computations for a short flume at this point could be safely based on a value of $n = 0.015$, as for lined concrete ditches. A long flume should be designed more accurately,

for economy; a textbook on hydraulics (1, 5, 7) will provide a better n value. Normally there would be a smaller value of n than 0.015 for a smooth metal or wood flume. If so, a smaller flume could do the work than would be indicated using 0.015. For a short flume the economy might not be worth the trouble of looking up the text.

The preceding discussion of design and construction of ditches has purposely omitted the structures generally present. These devices cannot be neglected in design; they sometimes act as partial dams, backing up the water and changing the slope of its surface in the ditch a considerable distance upstream. This change gives rise to a new set of conditions to control the flow. The gradient is lowered, or the slope is less; and the depth of the flowing stream is increased. The wetted perimeter and the cross-section area have changed; and the hydraulic radius has also shifted in value. If such structures have a moderate effect on water levels in the ditch, no trouble may be experienced in a ditch designed to operate without them. The freeboard from the water surface to the top of the banks must be adequate—a strong argument for providing such freeboard in constructing a ditch.

As was mentioned earlier, the control devices in ditches are not new, but the name applied to the same structure varies in different localities. The general design details of these various controls will be described hereafter on the basis of what they are supposed to accomplish, and some of the names will be given.

The ditch needs protection from one thing—namely, the hazard of washing out. This problem arises during normal operations and also from abuse or carelessness; and it is present with either lined or unlined channels. The lined ones can withstand higher rates of flow than the unlined, but neither can withstand overflow of the banks very long without experiencing a serious washout. Both types must be protected by taking special precautions at points where a side delivery is made to a field or into another ditch. Often such side outlets involve a sudden change in grade between channel water surface and that of the outlet. This available fall for the water sharply increases the velocity and the swirling erosive action of the stream. To avoid damaging the banks and to protect critical outlets, one must build permanent structures along the sides and across ditches.

SUPPLEMENTAL STRUCTURES FOR DITCHES AND CANALS

Permanence in a structure on a ditch depends upon the uses and abuses to which it is subjected as well as upon the strength and design details of the unit. Design assumes that some abuse must be met and absorbed and that the construction is heavy enough for safety. The trampling of animals can destroy the ditch bank, permitting a washout that will wreck the finest structure. To pry stuck gate slides will rack and loosen the frames of wooden structures, eventually causing leaks and washouts. High-velocity flows are ruinous. Failure to maintain the structures will result in serious deterioration. All these are abuses. The amount of protection to be specified in design, both for mechanical strength and for prevention of washouts, depends upon the soil characteristics

near the structure. If the soil is loose or sandy, washing will be more likely; the structure must have more strength and greater washout protection than in soils having greater natural stability. Gate openings smaller than are required may fail to pass all the water. When this happens, the water backs up in the ditch, causing overflows and washouts upstream, or at the structure, or in both

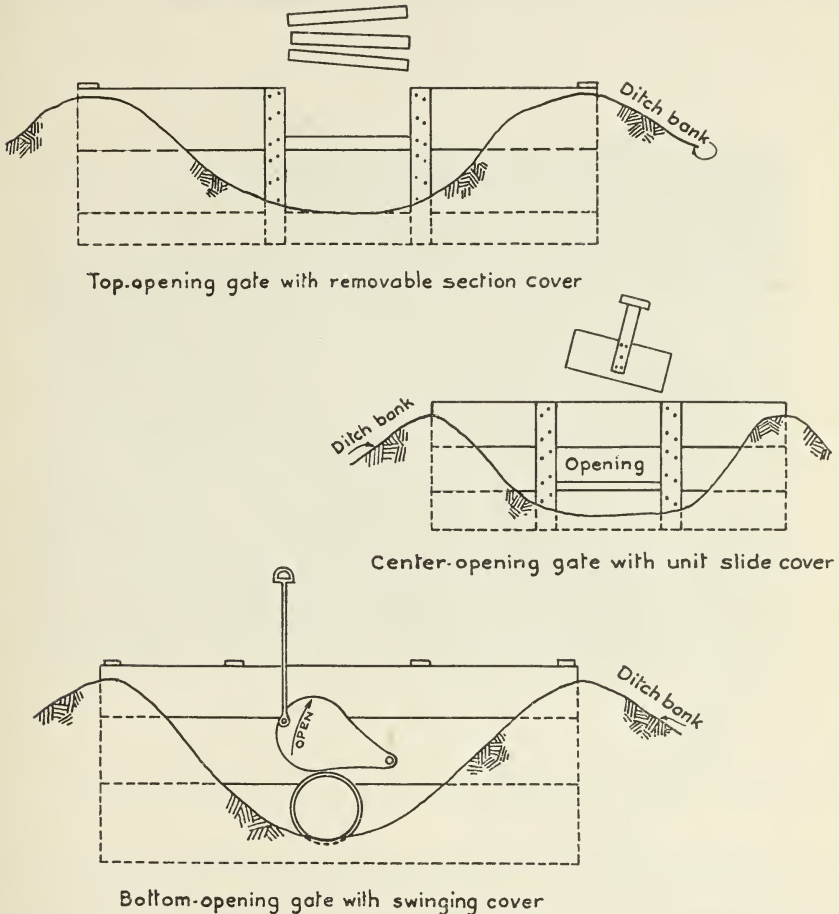


Fig. 19.—Three simple ditch structures, showing different types of opening.

places. Structures made too lightly will warp and sag, permitting leaks and eventual washouts. Such problems must be solved.

Normally a ditch structure is a bulkhead containing an aperture that can be opened and shut at will or through which the flow can be somehow controlled. This aperture may be located at the top, the center, or the bottom of the face of the structure. The movable cover or gate may slide as a unit, or be removable in sections, or swing in an arc across the opening (fig. 19). The simple bulkhead becomes more useful if supplementary framing is added to protect the ditch further, or to make the structure more flexible in its use, or for

both reasons. The name of the structure will change as different framing is added: a new name will identify some special capacity of the unit as constructed. The simple bulkhead could be used unsupplemented for every control operation if the natural weakness of the ditch, the washout hazard, did not necessitate more protection. For example, a simple bulkhead with an aperture at the top could successfully divert water into a flume or pipe while holding the

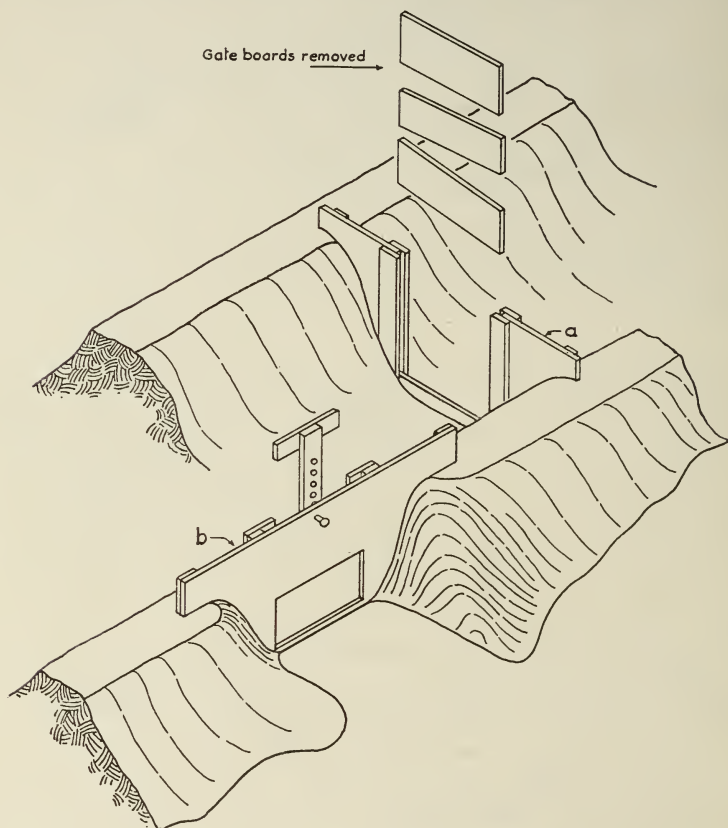


Fig. 20.—Typical installation of two single-wall wood gates in a ditch.

level in the ditch fairly high. If this same structure had to keep the water levels up in the first ditch and if at the same time water poured over the crest of the opening and down to an adjoining field or into a ditch, trouble would result. The overpouring stream would increase in velocity in falling, and the energy thus developed would be dissipated where the stream landed. With the simple bulkhead the stream would fall on earth; and the earth would be washed away, forming a hole into which the structure itself would eventually fall. To prevent this potential failure, an apron of impervious material (wood, concrete, metal) could be built below the structure at, or a few inches below, the normal ditch bottom. The simple structure holding the water levels in the ditch might be called a "check gate": it halts the flow in the ditch till the water

levels are raised a predetermined amount. It might be called a "turnout gate" or a "field gate," while acting at times as a "check." Given the apron, with the ability to stand up in use while the water dropped down on the apron, it could be called a "drop gate" or a "check drop"; the latter name shows its dual capacity. Where such a gate is the start of a lateral ditch, the term "head gate" might also be applied because the structure is, in fact, the head gate of that lateral. If the gate cover in any structure is raised partially, permitting the water to flow under the cover, the flow is "undershot," and the gate might be described as an undershot gate. As the foregoing illustrates, the name indicates

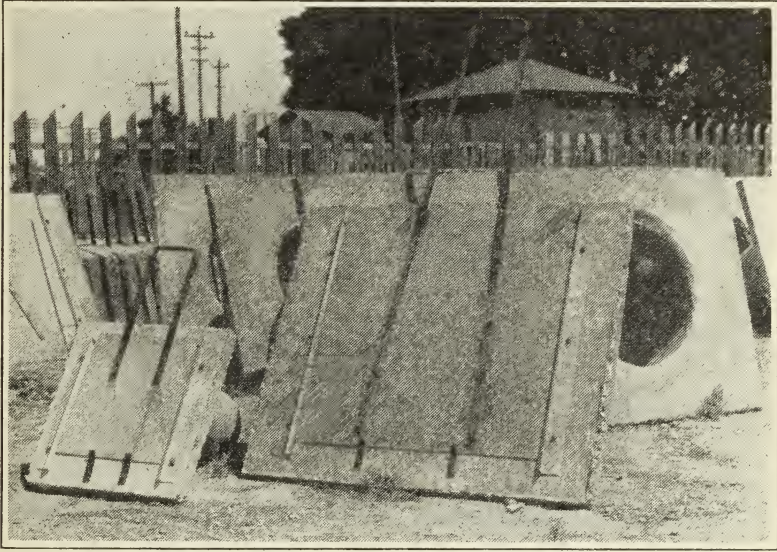


Fig. 21.—Prefabricated single-wall concrete gates ready for installation.

the function; but there is really only one basic structure to which supplemental framing is added to give the required protection under special conditions.

As these illustrations show, one may discuss the ditch structure by starting with the simple bulkhead type, adding to that type till the most complicated structure is evolved. On this basis one can explain the uses and limitations of each stage in the evolution of the structure.

Single-Wall Structure.—Figure 20 illustrates two typical single-wall structures located near each other on a ditch. (Figure 21 shows a concrete single-wall gate.) The one labelled *a* (fig. 20) spans the ditch, whereas *b* lies across an opening in the bank; *a* has a full opening, whereas *b* opens near the bottom. Each could replace the other. The gate *a* has the following uses: (1) A stop, to prevent flow in the ditch beyond *a*. (2) A check to back the water up a little so the flow through *b* will be satisfactory. (3) A check drop, to back water up in the ditch so some flow will pass through *b* while the excess passes over the removable gate boards to the downstream side of the ditch. Unless the drop

in this case is very limited (an inch or two), damaging erosion will result. (4) An undershot check, with the same purpose as in (3), but with excess water flowing under the gate boards on *a* instead of over. Possible erosion might result if water rises very high on the upstream side of *a*. (5) Unobstructed flow of the water when the gate is wide open.³

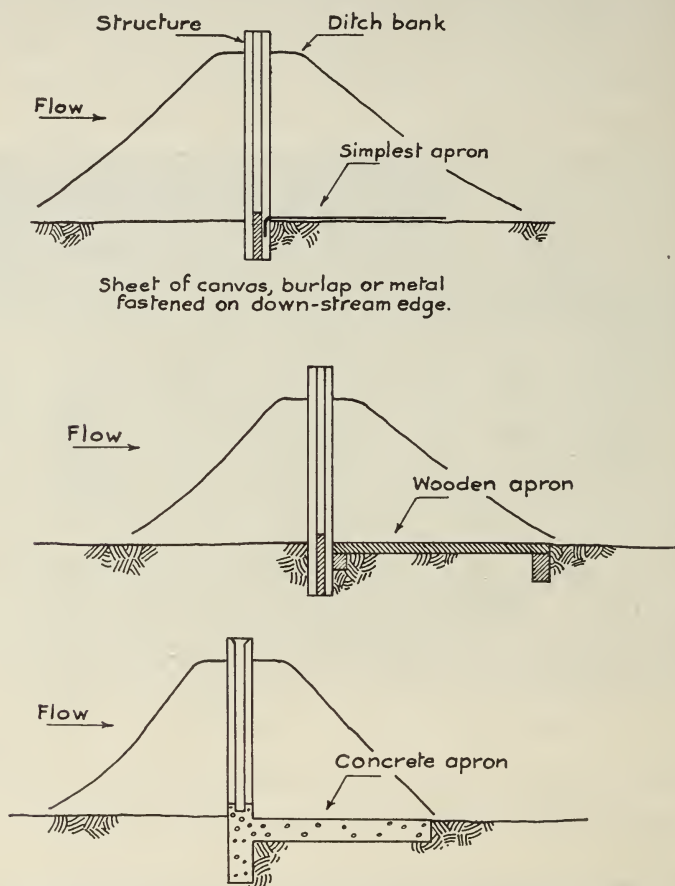


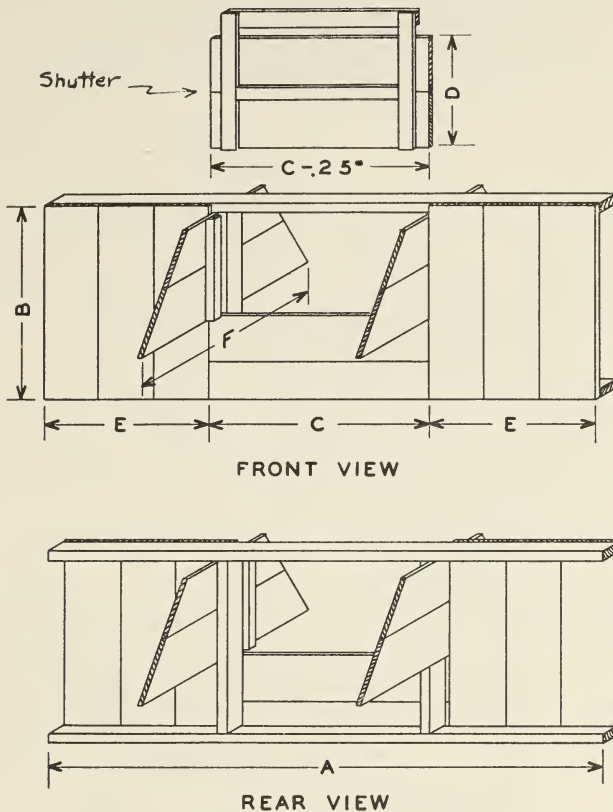
Fig. 22.—Three typical apron installations below single-wall gates.

The gate shown at *b* has the following uses: (1) A stop, to prevent flow if none is desired. (2) A turnout to release water to an area of use. (3) An undershot check permitting undershot release of water along the bottom of the gate boards while a head of water stands above the opening of release. (This use, however, is limited.)

As may be noted in figure 20, both *a* and *b* extend into the bank a consider-

³ The opening *a* or *b* must be large enough to permit the required flow to pass through quietly; otherwise erosion will result downstream. The tabulation for permissible velocities in ditches for various types of soil (table 1) gives the limiting velocities for the several soil types. Thus, if the soil type will accommodate 1 foot per second velocity and if we desire to pass 900 g.p.m. or 2 c.f.s. through *b*, the size of opening *b* should be not less than 2 square feet—or, for safety, $2\frac{1}{2}$ to 3 square feet.

able distance along both sides and at the bottom of the structure. This arrangement is necessary both for support and as a seal to permit the earth to close off flow from one side of the wall to the other. Because of the simple, light wall



RECOMMENDED SIZES						
C.F.S.	A	B	C	D	E	F
3	7'0"	2'6"	3'0"	1'6"	2'0"	3'0"
6	9'0"	3'0"	3'0"	2'0"	3'0"	3'6"
9	10'0"	3'0"	4'0"	2'0"	3'0"	3'6"
12	11'0"	3'6"	5'0"	2'0"	3'0"	3'6"
16	11'0"	4'0"	5'0"	2'6"	3'0"	4'0"

Fig. 23.—Single-wall gate with side walls only for protection of banks.
The table shows dimensions required for various flow capacities.

construction, these gates are limited to rather narrow ditches (4 to 6 feet of unsupported space is maximum). They are not strong enough for long spans, but would bow under load, jamming the gate boards fast, and would probably be ruptured by the resulting pressure. More complex designs gain strength that permits their use in large ditches.

There are many shortcomings of a and b : (1) as already noted, they are

structurally weak; (2) the wing walls and bottom of the structure, though imbedded in the ditch body at least $1\frac{1}{2}$ times the height of the outlet hole, are subject to seepage or leakage, which brings disaster; (3) spilling over of water from one side of either structure to the other side will cause serious erosion; and (4) downstream surfaces are subject to erosion.

Single-Wall Structure Plus Apron on Downstream Side.—The addition of an

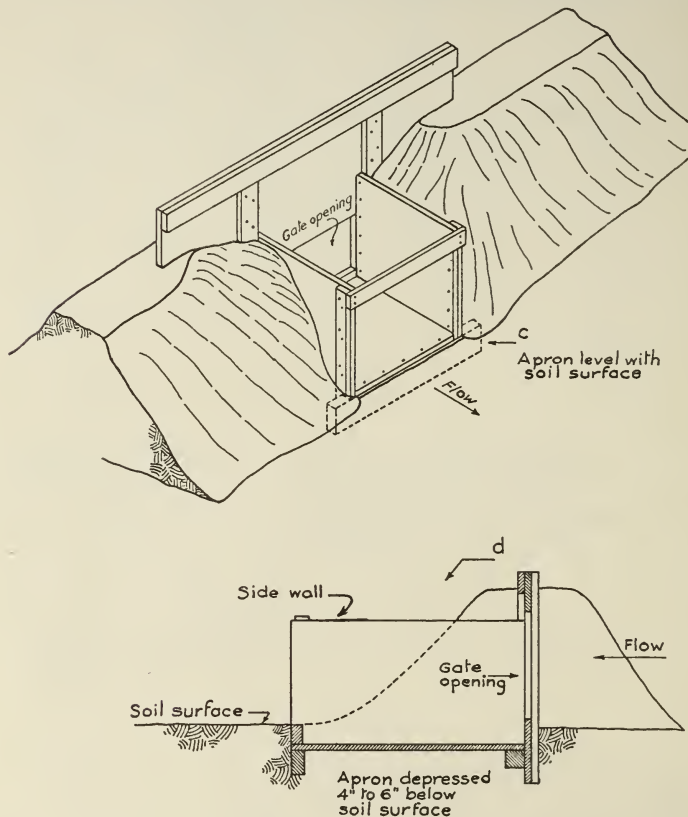


Fig. 24.—Two simple gates with apron: upper has apron flush with ground; lower has apron depressed below the soil surface.

apron is the simplest improvement that can be made to the single-wall structure. The apron (fig. 22) increases the safety of the unit. The installation can be safely used as a drop for short periods or for small flow for indefinite periods. The overpour, as it lands on the apron, rushes both downstream and at right angles toward the ditch banks. As a result, the banks close to the apron are subject to the swirling action of those side-thrusting parts of the stream. This is a weakness and renders the apron by itself only partially effective. Aprons can be made of almost any convenient material, ranging from burlap sacks to concrete. The addition of side walls to the apron is the obvious next step. (Figure 23 shows details of structure with side wall only added.)

Single-Wall Structure Plus Apron with Side Walls on Downstream Side.—Figure 24 illustrates variations in the use of the apron under the nomenclature *c* and *d*. Although they are actually the same apron, one (*d*) is depressed 4 to 6 inches below the soil level at the outlet to make a pond just below the gate opening that can help to absorb the energy if water is dropped over the top of the gate boards, thus helping to prevent erosion. The side walls on either *c* or *d* assist in protecting the bank of the ditch from the splash and eddy of the

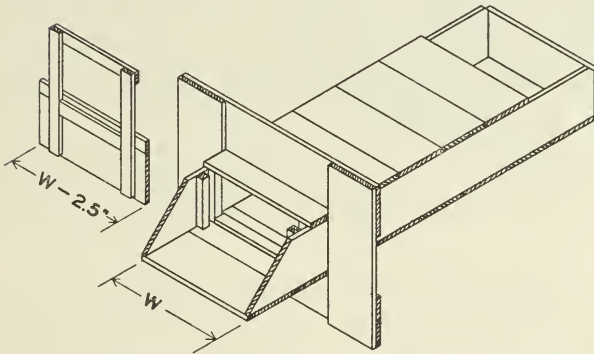
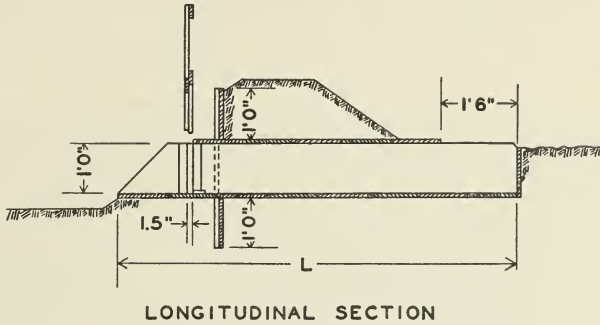


Fig. 25.—Gopher-type gate. Single wall plus a boxed-in apron.

water. Of the two aprons, *d* is the more effective. As with the simple wall, the limitation of these structures is the frailty of the wall itself and the likelihood of leakage. The apron framing does add some strength; but if leakage starts, this added strength is of little value. The side walls on the apron make that addition a short-length flume.

Gopher-Type Turnout or Field Gate.—This special adaptation of the single-wall structure is used where delivery must be made in the field at a short distance from the ditch and with the least possible turbulence—a feature desirable where sandy soil prevails and where washing may be severe. Details are given in figure 25. The gate is a single-wall structure with a completely boxed apron extending through the ditch bank discharging at, or slightly below, the surface

of the field being served. Its chief uses are as follows: (1) a stop when no delivery to the field is desired; (2) an undershot check if a controlled part of the ditch flow is to be taken from the ditch; (3) a device that gives a quiet welling flow upward onto the field with little agitation; (4) a structure that does not cut the main ditch bank by wing walls; and (5) a protected inlet due to extension of apron a short way into the ditch.

The gopher-type gate has been utilized only where special precautions are desirable to prevent washing. Considerable savings in soil otherwise lost by erosion would result, however, if it had more frequent use. One objection to it is the maintenance, in the field, of an open hole into which stock or farm

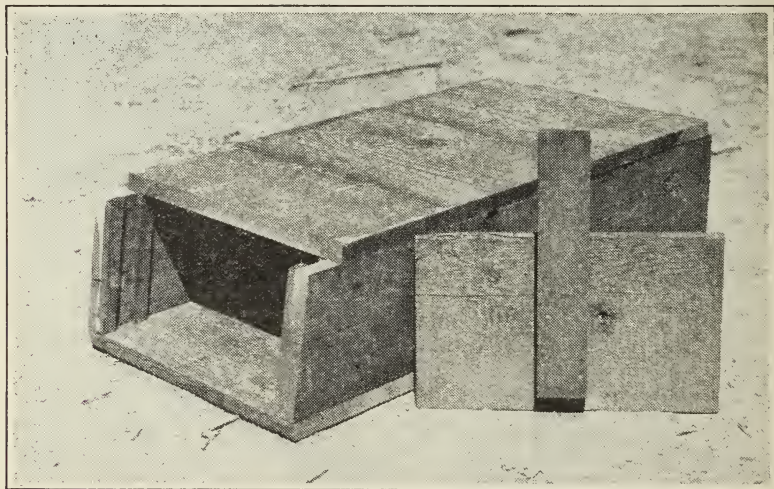
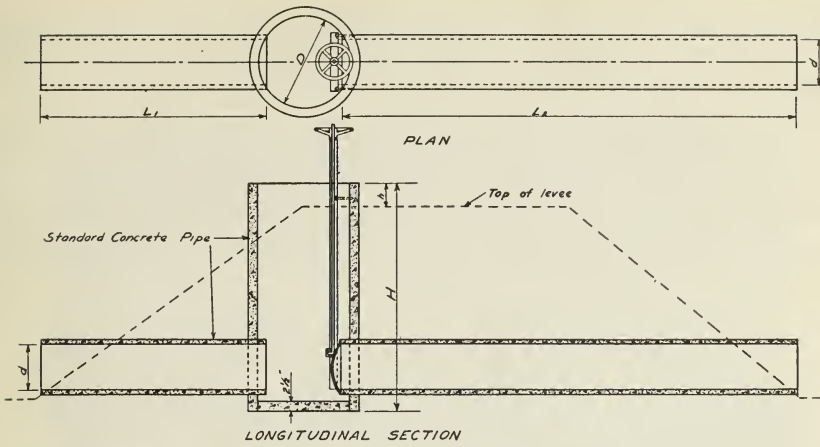


Fig. 26.—Simple form of gopher-type gate having no wing wall.

equipment may fall. In addition, the shallow soil cover (if any) makes the apron subject to damage if crossed by heavy farm implements or large animals. Figure 26 shows a simpler form, extending through the ditch bank only. Adaptations to concrete (fig. 27) permit stronger construction, with the outlet in the field closed by a disk-type valve.

Double-Wall Structure with Connecting Flume.—To remedy the shortcomings of the simple wall-type gate, the double-wall gate is used with a flume joining the two walls. This construction adds much mechanical strength and almost eliminates leakage. Figure 28 shows a typical gate. In it the two simple walls tied together by the flume and braced by the earthen fill between their wings can be made firm and watertight even in very loose soil. Where there is still hazard of leakage, the length of the flume is increased, widening the distance between the walls. This structure may serve as (1) a stop or a check; (2) a drop, if spill is into flume; (3) an undershot or overshot delivery gate, delivering preferably into a flume; (4) a turnout or field gate replacing *b* in figure 19 in general, but especially in loose soil; (5) a head gate; (6) a gate across large



d	D	d	D
8"	24"	16"	24"
10"	24"	18"	30"
12"	24"	20"	30"
14"	24"	24"	36"

NOTES ON DIMENSIONS
 H - Standard heights, 4'0", 5'0" and 6'0"
 h - From 6" to 12"
 L_1 - Usually 5'0", Minimum 2'6"
 L_2 - As required. Usually 7'6", 10'0" or 12'6"
 Pitot tube measurements are made in pipe upstream from standpipe.

Fig. 27.—Application of concrete pipe to gopher-type gate.

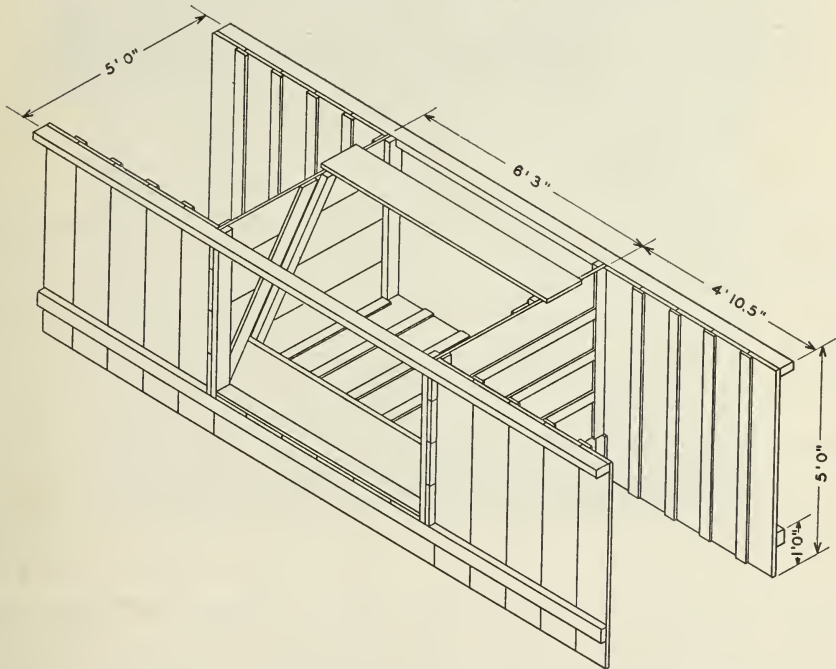


Fig. 28.—Double-wall structure with gate in connecting flume. The dimensions show typical construction proportions.

spans where the simple gate would fail; and (7) a device into which double gates can be put, one behind the other, with earth between them to assure tightness.

Evidently, therefore, this type has almost universal application. It is especially useful for unlined ditches. Variations have the wing walls set at an angle other than 90 degrees to the flume line (fig. 29) or wing walls of differing length. The angle-set wing walls create a throat at the start and end of the flume, reducing turbulence and erosion. The varying length in these walls adapts the structure to each situation without waste.

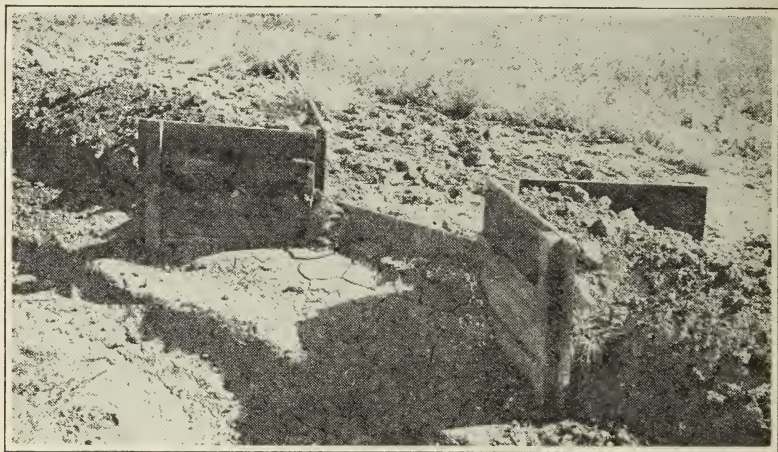


Fig. 29.—Double-wall gate. The throat in this gate is shown reduced to the minimum.

All gates, regardless of their use, are variations of those already described. Not all the modifications can be mentioned; but with the description and illustrations given, anyone can devise a gate to satisfy special requirements. Since, however, some of those detailed will meet almost any condition, individual planning will usually be needless. One phase in the placing of any structure has been referred to without details—namely, the tamping of earth about the walls to form a water seal and, at the same time, to support them. Where a flume or an apron accompanies a single-wall or multi-wall structure, the bottom cutoff wall must be placed first, and the excavated earth tamped about it. This tightly packed earth must be carefully graded and trimmed to receive the remaining structure. Tamped earth must have the damp-soil characteristics already described as necessary for ditch banks. Because small leaks through soil become large ones in time, careless refilling of excavations may result in destruction by washout when the structure is in use. Obviously, careless procedure here can ruin a good design. Special attention must be given newly installed structures to make sure they are securely sealed in the earth banks.

Besides the true structure, there are several devices used as gates as well as

special adaptations of gates and other contrivances for controlling water in and delivery from ditches. Most of these units have a limited application.

A serviceable gate can be made from a section of large-diameter sewer tile

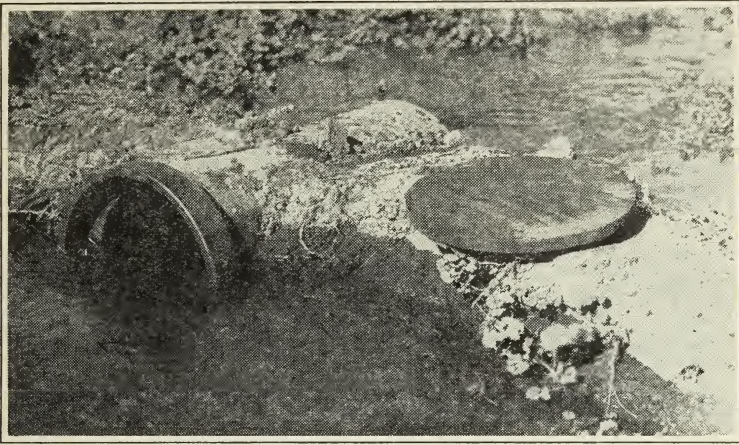


Fig. 30.—Sewer tile used as a gate on the side of a ditch.

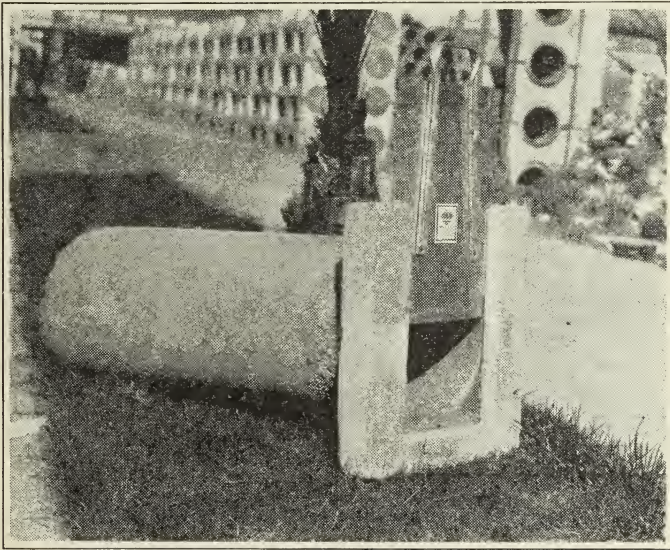


Fig. 31.—Concrete pipe with gate cast at one end.

or from a large concrete pipe, a removable wood disk being used as a seal when no flow is desired. Such a gate (fig. 30) may be installed where ditch banks are relatively impervious, so that seepage is not a problem along the outside surface of the pipe. Big pipes (see also fig. 31) pass large flows of water for flooding operations; but for furrows several small streams are better. A series of small galvanized-iron pipes or rectangular or other milled-bore wooden tubes with

TABLE 3
FLOW THROUGH REDWOOD FURROW TUBES OF DIFFERENT
DIMENSIONS AS SHOWN IN FIGURE 32*

Shape of tube	Length	Diameter of cross section	Depth of water over inlet of tube	Flow
	<i>feet</i>	<i>inches</i>	<i>feet</i>	<i>gallons per minute</i>
Circular.....	2	$\frac{3}{4}$	0.64	5.22
Circular.....	2	$\frac{3}{4}$	.206	3.50
Circular.....	4	$\frac{3}{4}$	.617	4.22
Circular.....	4	1	.227	2.75
Circular.....	2	1	.598	9.17
Circular.....	2	1	.184	5.62
Circular.....	4	1	.590	7.78
Circular.....	4	1	.194	5.22
Circular.....	2	$1\frac{1}{8}$	.578	12.30
Circular.....	2	$1\frac{1}{8}$	.156	7.62
Circular.....	4	$1\frac{1}{8}$	.573	10.21
Circular.....	4	$1\frac{1}{8}$	.175	6.24
Flattened ellipse.....	2	1×2	.517	19.69
Flattened ellipse.....	2	1×2	.135	10.90
Flattened ellipse.....	4	1×2	.526	18.08
Flattened ellipse.....	4	1×2	0.146	10.07

* Discharge of tube passes freely into air (not submerged), and the tube is horizontal.

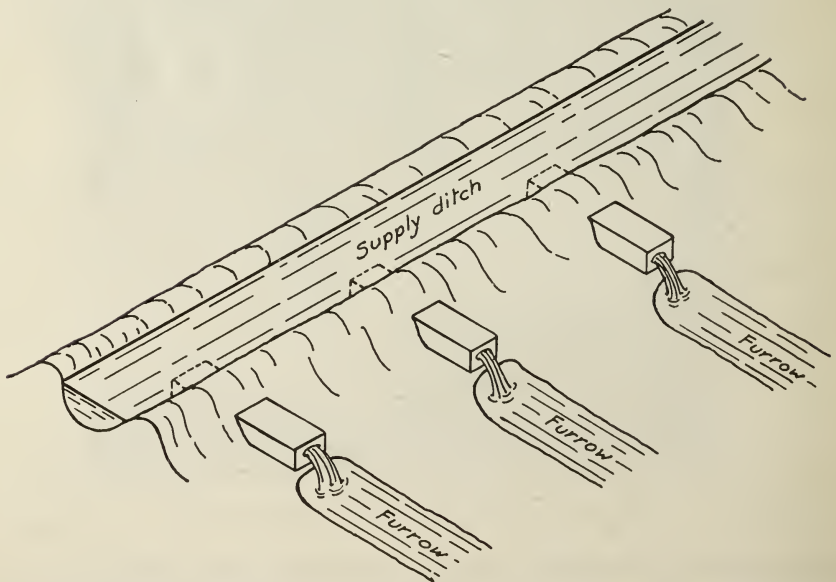


Fig. 32,—Wooden furrow tubes in use. The openings are usually made elliptical or round,

proper movable gates can serve a group of furrows all at one time. Such small furrow feeders or furrow tubes are shown in figure 32. As used, these tubes pass through the ditch bank—a weakness, since each separate tube may permit leakage and washout. Table 3 gives the capacities of several shapes and sizes of furrow tubes. One can substitute for the tubes a series of small light-weight siphons, 1 to 2 inches in diameter. These are filled by submerging in the ditch. Then one end only is withdrawn, sealed by the palm of the hand; and, when this outside end is lower than the water in the ditch, the hand is removed. Flow commences at once; and a man can control 20 or 30 siphons easily, either as furrow feeders or for flooding operations. He picks them up when the irrigation is complete and moves them to the next location, where the cycle is

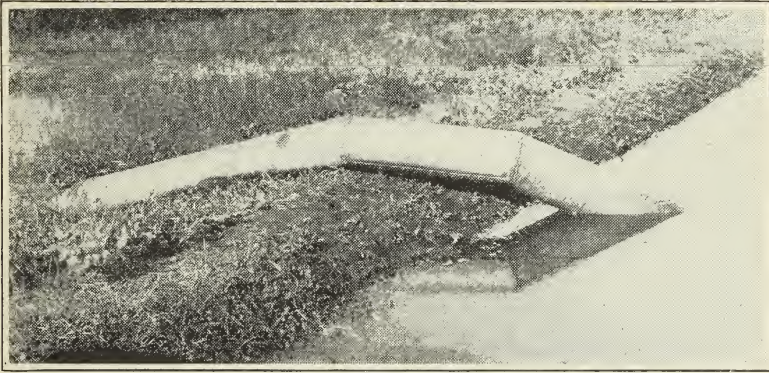


Fig. 33.—Six-inch galvanized iron siphon in operation, showing flow from ditch (right) to field (left).

repeated. Occasionally, larger 6- to 8-inch siphons (fig. 33) are used; but they are extremely cumbersome. The advantage of the siphon is that ditch banks are unbroken, yet the water is available at any time at any area of use. In addition, one set of siphons will serve for all the irrigation. A labor saving may be made possible by the use of siphons in some locations.

Often it is desirable to use a ditch in sections, filling new parts lower down as the irrigation progresses. A series of permanent structures for this purpose would be costly; but a portable dam or stop in the form of a canvas on a frame or metal panel, called a tappoon, that can be driven into the earth and across the ditch, can be used repeatedly to control the ditch flow. These devices (fig. 34) are applicable to earth ditches. To permit the same control on lined systems, slots for such gates could be cast into the faces of the channels at any desired intervals. One set of gate boards could then be used at any point where slots are provided. Under some conditions, permanent structures serving as tappoons are placed in earthen ditches where the grade conditions either in the ditch or on the land necessitate their use. Two such applications are detailed in figure 35.

Since an open ditch is an obstruction, a passageway for farm machinery

must be provided. Two procedures are possible. The first is to build a bridge over the ditch; the second to run the ditch under the road. This latter method is common where ditches cross highways and the road cannot be graded up over the ditch banks to permit the use of a bridge. Figure 36 illustrates a typical bridge. Most ditches are not too wide to permit use of a simple flat span, as illustrated. If a center pier, or piers, is added, two or more flat spans

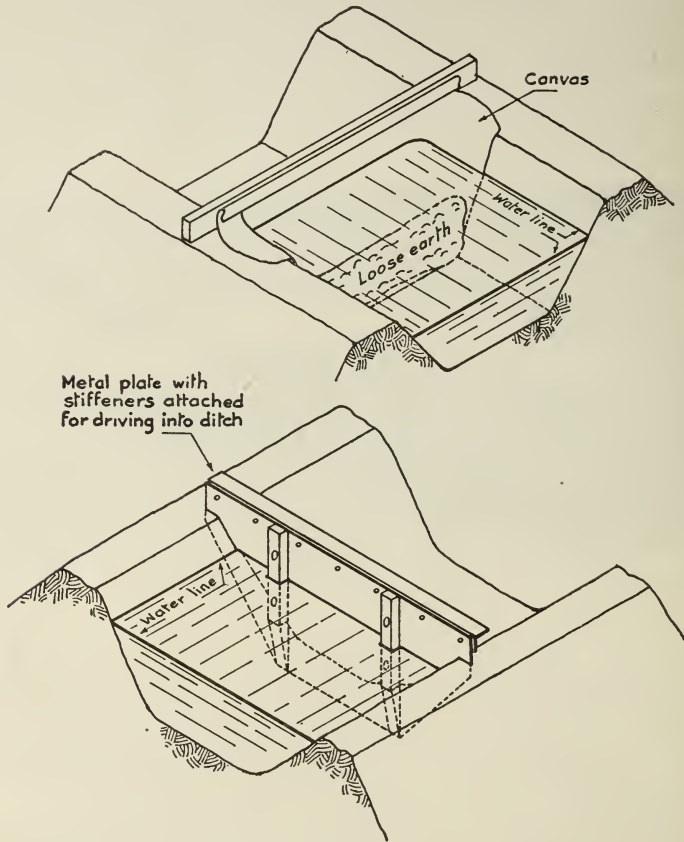


Fig. 34.—Two types of tappoon shown in use.

of this type may be used. Larger, more complex permanent bridges can be built; their construction will probably be more economical if a competent designer is given the assignment. The bridge shown in figure 36 can be adapted to most farm requirements. In many locations the road grade must be raised from some distance away toward the bridge to keep the bottom of the span above the high-water level in the ditch. The ends of the bridge should rest on a solid footing or foundation to protect the banks, which when wet might not support the ends of the bridge.

The operation of running the ditch under a road is that of transferring the flow into a depressed pipe. Such a pipe is called an inverted siphon (see fig. 37

for details). It need not necessarily be round, but may be rectangular or have any other easily constructed cross section. The size necessary may be calculated as in the pipe-line problems given later in this paper.

One should design the inlet and outlet of a siphon so that the water will enter and leave with a minimum of turbulence. A tapered inlet throat and a similar outlet with the large end of the taper toward the ditch will provide

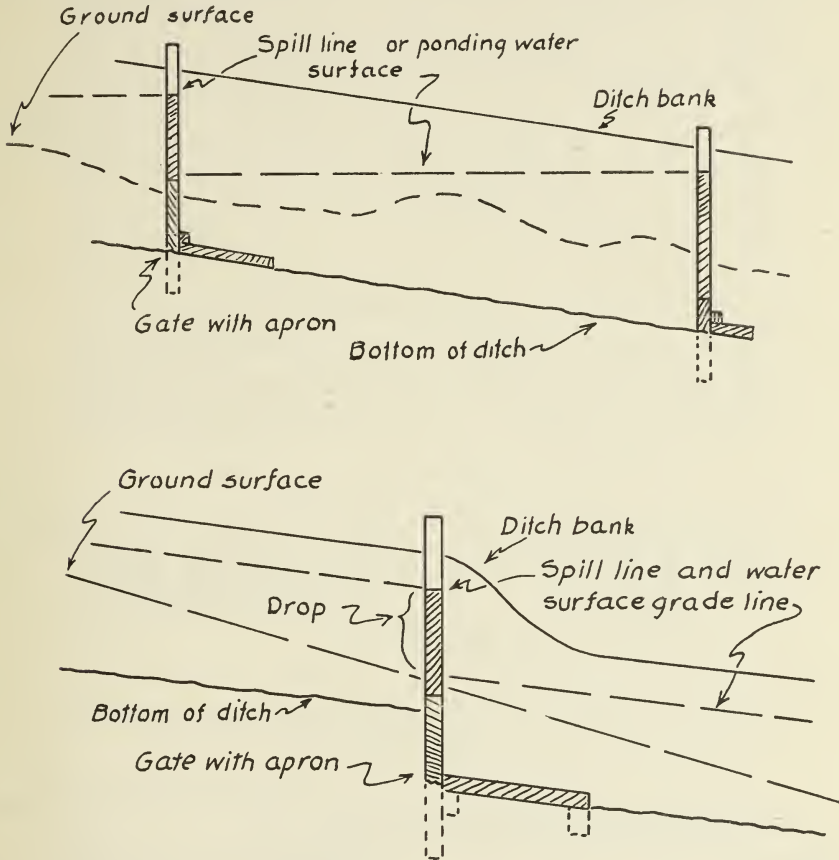


Fig. 35.—The upper drawing shows use of a simple-wall structure as check or stop gate to pond water behind it in order to raise the water level in the ditch above adjoining land to be watered. Its use is similar to that of a tappoon with a drop overflow; the lower drawing shows the simple structure used as a drop gate along a ditch whose slope is the maximum permissible for the soil type. The ground surface slopes more sharply than the ditch; so the drop must be used to lower the ditch line, keeping it closer to the ground-surface elevations.

desirable conditions. Figure 37 shows these throats. Their design is not critical: any shape that produces a gradual change in dimensions from the ditch section to the pipe size will work. Economy in construction limits the length of the throats. As shown in the figure, the pipe itself should be completely submerged at the small end of the throat; otherwise the flow of water to the siphon will be limited to the amount that can enter the partly filled pipe inlet, and

most of the value of the throat will be lost. A covering of at least 18 inches of earth is desirable to protect the top of the siphon from damage by heavy traffic. Such structures are sometimes called culverts.

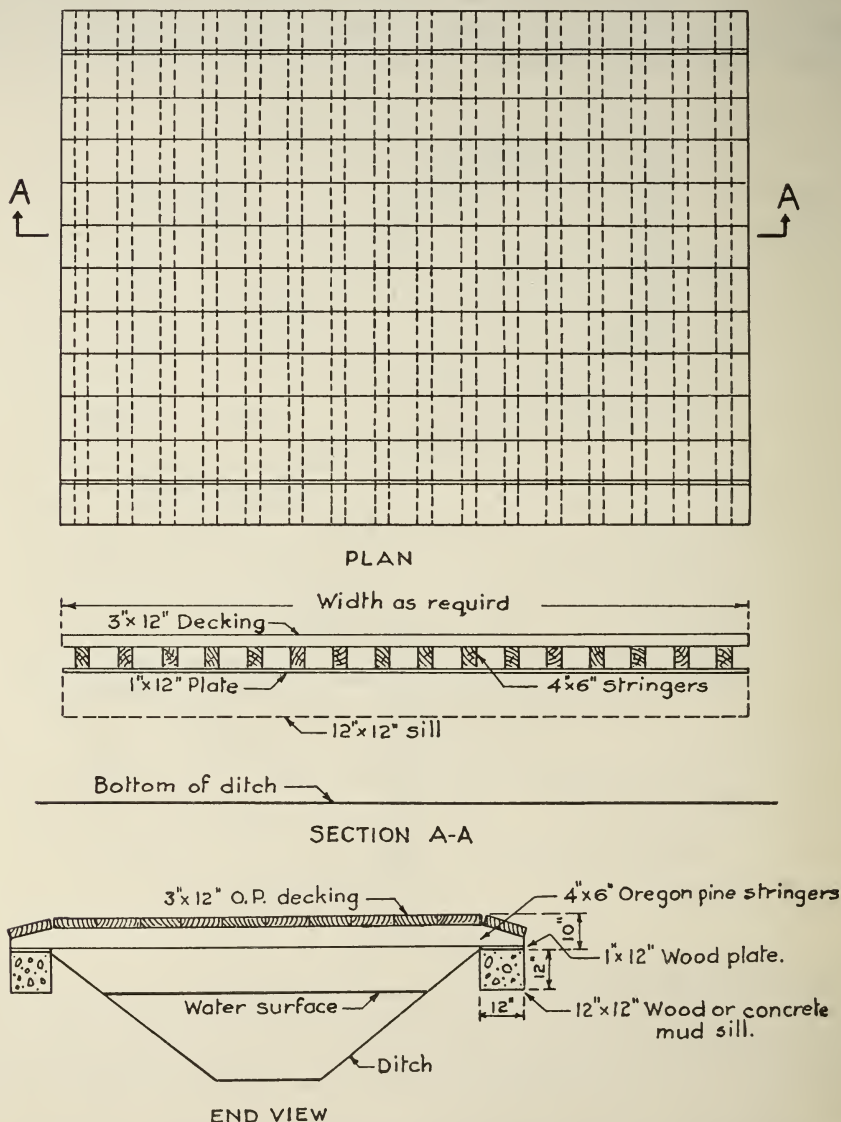


Fig. 36.—Details of simple wooden bridge to span an irrigation ditch. Redwood is most suitable for wood mud sills.

Structures Used as Meters.—Certain devices may be adapted to use as meters for indicating the flow. Where water is being sold, this feature is especially useful. Suitable meters are those in which changes do not occur during use. As a rule, therefore, they are confined to installations having permanent con-

struction like that afforded by concrete and steel. Permanence is necessary so that the meter will remain consistent—a condition not found in wooden structures where the boards warp, come loose, or are torn off. One can test any structure to obtain a rating curve that will be useful for a short period. Since, however, a test is difficult to make, it is not worth while unless the results

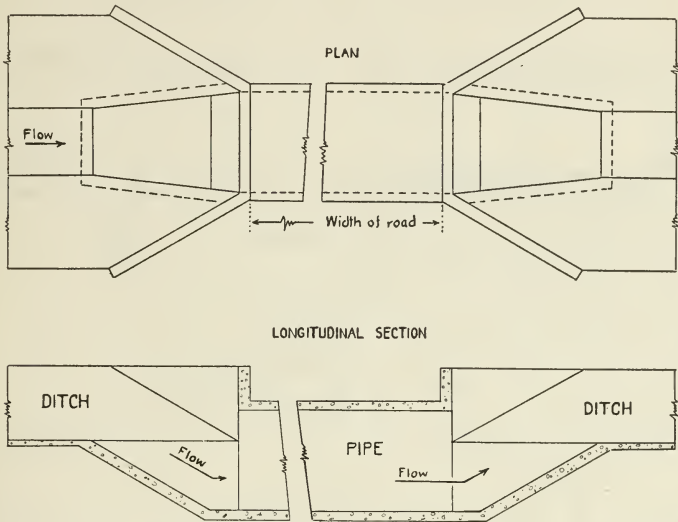


Fig. 37.—Inverted siphon used to pass ditch flow under road or similar obstacle.

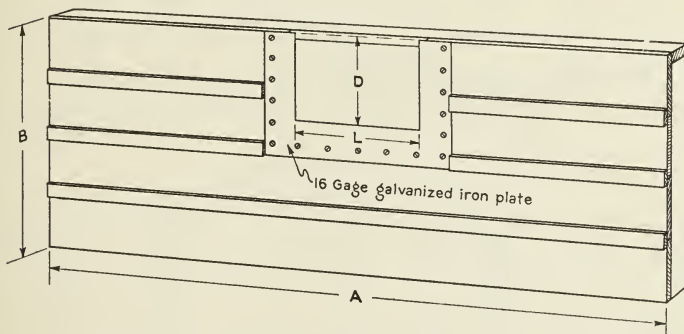


Fig. 38.—Single-wall structure with thin metallic edges on upstream face of opening, for accurate measurement of water.

are good for the life of the structure. A metal strip may be put on the edges of a wooden structure (fig. 38) to make a full-contracted weir opening, and many of the installations already described may be fitted to permit their use as meters. To serve thus, they should follow a standard design, duplicating a tested device for which tables of flow are available. California Agricultural Experiment Station Bulletin 588 (8) gives performance tables and describes many forms of measuring devices used in ditches. Construction details must

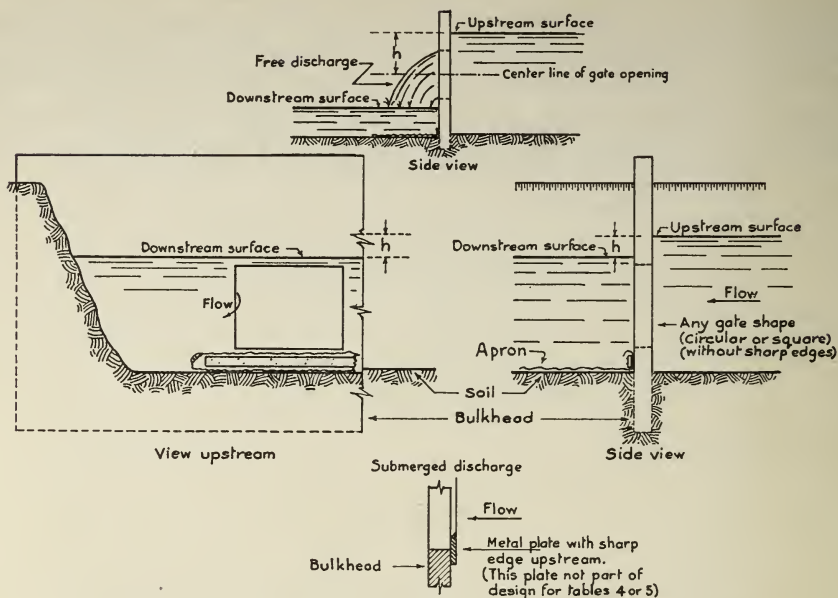


Fig. 39.—Gate for metering water without use of sharp-edged facing.

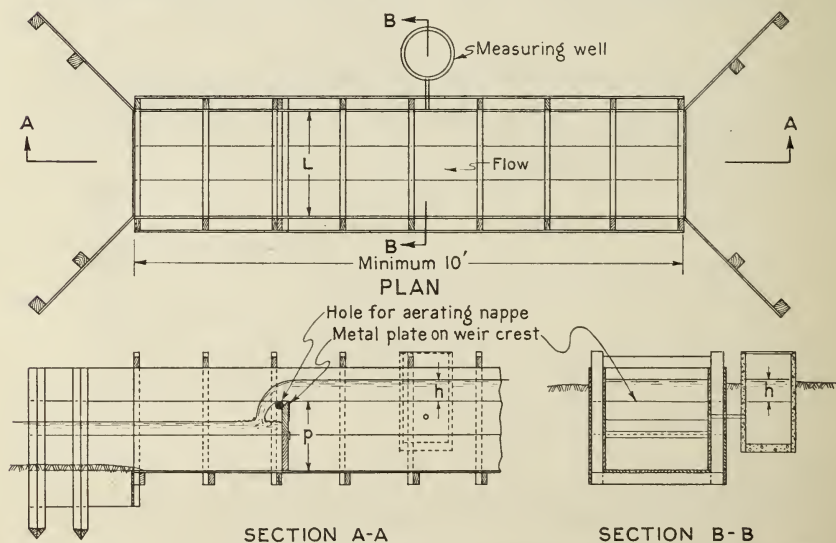


Fig. 40.—Gate in flume used as metering unit. Direct flow through flume to gate prevents water from curving inward in passing through the gate; this eliminates "end contractions."

be carefully reproduced so that behavior may duplicate that of the standard devices. In contrast, the average field gate may be roughly put together so that no two supposedly identical structures have exactly the same characteristics. Nevertheless, installations of similar general shape will behave roughly alike. Figures 39 and 40, with accompanying tables 4 and 5, give the approxi-

TABLE 4

APPROXIMATE FLOW THROUGH AN ORIFICE GATE, AS IN FIGURE 39, WITHOUT SHARP
ENTRANCE EDGES BUT WITH SOME CONTRACTION

Area, in square feet†	Cubic feet per second for various indicated heads, in feet (<i>h</i> in figure 39)*									
	0.2 ft.	0.4 ft.	0.6 ft.	0.8 ft.	1.0 ft.	1.2 ft.	1.4 ft.	1.6 ft.	1.8 ft.	2.0 ft.
0.2.....	0.59	0.83	1.02	1.18	1.32	1.44	1.56	1.66	1.77	1.86
0.4.....	1.18	1.66	2.04	2.36	2.63	2.88	3.11	3.33	3.53	3.72
0.6.....	1.76	2.50	3.06	3.54	3.95	4.32	4.67	4.99	5.30	5.58
0.8.....	2.35	3.33	4.08	4.72	5.26	5.76	6.23	6.66	7.06	7.44
1.0.....	2.94	4.16	5.10	5.90	6.58	7.20	7.78	8.32	8.83	9.30
1.5.....	4.41	6.24	7.65	8.84	9.87	10.80	11.68	12.48	13.24	13.95
2.0.....	5.88	8.32	10.20	11.79	13.16	14.40	15.57	16.64	17.66	18.60
3.0.....	8.83	12.48	15.30	17.68	19.74	21.60	23.35	24.96	26.49	27.90
4.0.....	11.77	16.64	20.40	23.58	26.32	28.80	31.14	33.28	35.32	37.20
5.0.....	14.71	20.80	25.50	29.47	32.90	36.00	38.92	41.60	44.15	46.50
6.0.....	17.65	24.97	30.60	35.37	39.48	43.20	46.71	49.92	52.98	55.80
7.0.....	20.60	29.15	35.70	41.25	46.05	50.40	54.50	58.22	61.80	65.10
8.0.....	23.54	33.29	40.80	47.16	52.64	57.60	62.28	66.56	70.64	74.40
9.0.....	26.50	37.45	45.90	53.05	59.20	64.80	70.50	74.85	79.18	83.68
10.0.....	29.42	41.61	51.00	58.95	65.80	72.00	77.85	83.20	88.30	93.00
11.0.....	32.35	45.80	56.10	64.81	72.38	79.20	85.63	91.52	97.13	102.30
12.0.....	35.30	49.97	56.12	70.72	78.96	86.40	93.52	99.84	105.96	111.60
13.0.....	38.25	54.12	56.63	76.61	85.54	93.60	101.30	108.16	114.79	120.90

* Quantity in cubic feet per second = $Ca\sqrt{2gh}$ where $C = 0.82$, where $g = 32.2$, the acceleration of gravity, and h = the head given at the top of the table above.

† To determine area of gates having circular cross section, square the diameter, in feet, and multiply by 0.7854.

TABLE 5

APPROXIMATE FLOW PER FOOT OF WIDTH, THROUGH GATE WITH NO END CONTRACTIONS,
AS IN FIGURE 40*

Head in inches (approximate)	Head in feet	Cubic feet per second per foot of width under the given weir heights (<i>h</i> in figure 40)						
		0.5 ft.	0.75 ft.	1.0 ft.	1.5 ft.	2.0 ft.	3.0 ft.	4.0 ft.
1¾.....	0.10	0.111	0.110	0.109	0.109	0.108	0.108	0.108
2¾.....	0.20	0.312	0.307	0.305	0.302	0.300	0.299	0.299
3¾.....	0.30	0.583	0.569	0.562	0.555	0.552	0.548	0.545
4¾.....	0.40	0.916	0.888	0.873	0.858	0.851	0.844	0.840
6.....	0.50	1.31	1.26	1.23	1.21	1.20	1.18	1.18
7¾.....	0.60	1.76	1.68	1.64	1.60	1.58	1.56	1.55
8¾.....	0.70	2.26	2.14	2.08	2.03	2.00	1.97	1.95
9¾.....	0.80	2.82	2.66	2.58	2.49	2.45	2.41	2.39
10¾.....	0.90	3.43	3.22	3.11	3.00	2.95	2.89	2.86
12.....	1.00	4.11	3.82	3.68	3.54	3.47	3.40	3.36
13¾.....	1.10		4.48	4.30	4.12	4.02	3.93	3.89
14¾.....	1.20		5.17	4.95	4.72	4.61	4.50	4.44
15.....	1.30			5.64	5.36	5.23	5.09	5.02
16¾.....	1.40			6.48	6.04	5.87	5.71	5.62
18.....	1.50			7.15	6.75	6.56	6.36	6.26

* Data from table 14, bulletin 588 (8).

mate flow through two rather typical forms of field gates. Using such tables, the designer can approximate the required size of opening needed to pass a given flow when a head loss is available. To be safe, he will build the opening a little larger than the tables indicate, for his design might not perform quite so well as the one upon which the tables were built. Sharp edges—that is, metallic faces with beveled edges downstream (fig. 39)—actually restrict the flow through such gates; and if these facings are present the openings must be made at least 1.33 times as large as in the tables. In any field structure not built strictly to some standard design details may vary in behavior 10 to 20 per cent

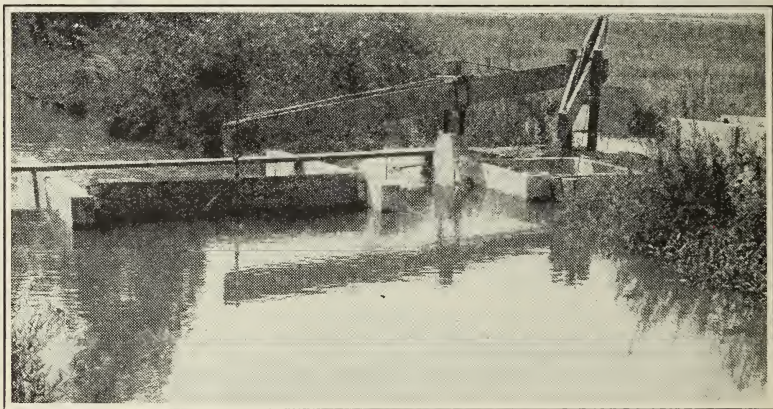


Fig. 41.—Automatic flow-control gate installed in canal.

from the figures given in these tables. Sharp edges of standard shape supply precise metering data. Complete details on such structures have been previously reported (8).

Automatic Gates.—The capacity to meter the flow to an area may simplify the operation of a conducting ditch that may be capable of handling a certain maximum safely. If conditions in the supply ditch are constant, one can set the gate for that flow and leave it. But since constant conditions are never encountered on some ditches, attempts have been made to put in automatic gates (used as head gates as a rule) that will vary their opening as the supply-ditch water levels vary. An effective automatic structure of this kind will save the operator much grief. It will serve also—though less effectively—as a meter. Figure 41 illustrates a typical automatic gate installation—float-operated lever that moves the gate board, the float being located in a sump, or a well, connected by a pipe to the main ditch. The float could be connected to the supply ditch or to the secondary ditch, whichever is intended to receive the control. Further refinements include electrical contacts on the float system, with an electric motor drive on the gate. These latter installations can be made to operate almost perfectly with widely fluctuating flow levels in the supply ditch, keeping constant the flow in the ditch being supplied. Most automatic gates without supplementary electrical equipment cannot be completely effective

during severe fluctuations in the supply-ditch water levels. Almost any slide gate can be made automatic according to the general principles pictured in figure 41.

Materials Used in Structures.—Various materials have been described and pictured in the figures in the preceding discussion of ditch or canal structures. Occasionally, one or the other material has been noted as best for a special use. Concrete and steel are, as might be expected, more permanent than wood. In localities where ditch grades shift and fields wash regardless of the care taken to prevent these changes, the concrete structure may be too permanent. Little can be done with a permanent structure when soil levels have receded or when

TABLE 6
DIAMETERS, AREAS, WEIGHTS, AND COST OF CONCRETE PIPE INSTALLED*

Diameter of pipe <i>inches</i>	Normal length of each section <i>inches</i>	Cross-section areas		Weight <i>pounds per foot</i>	Approximate cost, installed and backfilled, per foot (24-inch cover) <i>dollars</i>
		<i>square inches</i>	<i>square feet</i>		
6.....	24	28.28	0.1964	21	0.25 to 0.42
8.....	30	50.24	0.3489	28	0.27 to 0.42
10.....	30	78.50	0.5451	38	0.30 to 0.45
12.....	36	113.04	0.7854	48	0.35 to 0.53
14.....	36	153.86	1.0684	71	0.51 to 0.65
16.....	36	201.00	1.3947	91	0.75 to 0.79
18.....	36	254.34	1.7661	108	0.95 to 1.04
20.....	36	314.00	2.1804	128	1.25 to 1.35

* Pots, hydrants, stands, gates, and gate installations are all extra.

the unusual has occurred and levels have risen. A wood structure, on the contrary, can be remodeled somewhat if conditions change. These statements are not made to belittle concrete or steel structures, but to stress the fact that permanent structures are best where conditions remain constant.

UNDERGROUND PIPE DISTRIBUTION SYSTEM

Irrigation water is distributed successfully by underground systems using either steel or concrete pipe—more often the latter. Pipe offers certain advantages: fields unobstructed by ditches; a capacity to disregard grade within reasonable limits; elimination of open ditches to prevent the spread of weed seeds through water transportation; practical elimination of seepage and evaporation losses; and permanence. Its chief disadvantage is the initial cost, which is higher than for ditches of equal capacity. Unobstructed fields simplify cultivation and the movement of equipment, and save valuable land areas for productive use. Within reasonable limits, a pipe line can go straight over ridges and across swales, whereas a ditch must be run around such obstructions, unless expensive structures are built to avoid such detours. The elimination of seepage into porous soils from unlined ditches may pay for the more expensive underground pipe system. Evaporation losses from ditches, although

usually not great, may well be considered. The pipe system eliminates this loss. Most unlined ditches either must be remade often, under normal cultivation practice, or cleaned and reshaped occasionally. This work is eliminated by the subsurface pipe systems.

Most concrete irrigation pipe is bought from manufacturers who produce and stock large quantities of the standard sizes. Thus the consumer benefits by mass production, and the cost of forms is spread over a large volume of the product. Special equipment is required to produce even a few hundred feet of a given size of pipe in 30-inch lengths—the usual dimension; and only rarely



Fig. 42. Making concrete pipe in place in the field. Backfilling follows removal of outside forms immediately.

can the individual afford the time to make his own. Since the process has been described elsewhere (9) and can be observed in local yards throughout California, it will not be discussed here. Table 6 lists the various stock sizes of this pipe, its weight, and its approximate cost.

Ditching machines are usually employed to excavate the trench for the pipe. Then the trench is trimmed by hand to smooth out irregularities and to put the bottom on the exact grade, which may be a straight line, or a series of straight lines. The top of the pipe is kept safely below any anticipated cultivation operations (18 to 30 inches). An additional purpose in placing the pipe low is to reduce temperature and moisture changes, thereby reducing expansion and contraction. Pipe joints are connected by filling the recessed end of one pipe with a cement mortar and pushing the tongue end of the second pipe into the first, squeezing the mortar in the recess partly out of the joint. In

small pipes the excess mortar inside is wiped off smooth with a wet sack or brush, and the outside extrusion of mortar may be supplemented with more to make a ring or band around the joint. In large pipes, 24 inches or greater, a man often works inside, wiping and cleaning these joints; the outside of the joint is then banded as with smaller-diameter pipe. This work is often done as much as 24 hours after the joints have been made, to give settlement cracks a chance to form and the man the opportunity to seal them. This practice, to a great extent, eliminates leaks. Concrete pipe may be constructed on the job, using the ditch bottom as a form and using removable inside and top forms (fig. 42). Such a pipe is poured continuously and has no joints. In the larger pipes (30 inches or greater) so made, reinforcing may be incorporated as the construction progresses. The prefabricated pipe discussed above is the most popular. Since most concrete lines are laid by experienced crews from pipe yards, the finished job is usually satisfactory. The tabulation below shows the approximate cost of concrete pipe cast in the field as a continuous unit (sometimes called monolithic pipe, meaning all one piece). The smaller sizes are not made in the field.

<i>Size, inches</i>	<i>Cost per foot, dollars</i>
24.....	1.00 to 1.13
30.....	1.25 to 1.75
36.....	1.80 to 2.25
42.....	3.00 to 3.50

Flow through a pipe meets resistance; the rougher the pipe, the greater that resistance. The surface of concrete pipe resembles that of a concrete-lined ditch; and the same friction coefficient $n = 0.015$ can be applied to both. As a rule, steel pipe is smoother than concrete and would have a smaller value of n , or would pass more water for the same cross section. The pipe and the ditch are dissimilar in that the pipe is usually full, whereas the ditch has varying depth with varying flow. The hydraulic radius, R , in a pipe flowing full does not change, in contrast to the shifting value of R for open channels. The hydraulic radius of a full pipe equals

$$R = \frac{A}{P} = \frac{\text{area}}{\text{perimeter}} = \frac{\pi r^2}{2\pi r} = \frac{r}{2}.$$

Because the hydraulic radius is unvarying for any pipe size as long as the pipe is full of water, this R value need not be checked each time, and the flow-characteristic curves (figs. 43 and 44) can be simplified as compared with open-channel curve sheets (figs. 4, 5, and 6). Figure 43 gives the friction loss in ten sizes of steel pipe for a range of flows and is based, as noted in the figure, on Williams and Hazen's formula (10) for their coefficient of friction, $C = 100$. This is a conservative value for the friction that would correspond to about $n = 0.013$ in ditch design. Figure 44 gives the friction loss for the same ten sizes of concrete pipe, but supplies two curves for each pipe size from data published by Scobey (1, 11). The heavy or left-hand curve for each pipe size is

safe for the normal dry-mix pipe where the joints are not too well finished in the field. The right-hand or dash-line is satisfactory for wet-mix pipe, which is naturally smoother inside, and assumes that the field joints are well finished. The solid lines are safe for general design work and will be used in the following discussion. To use either chart (fig. 43 or 44) with the flow given, start at 3 c.f.s. on the bottom of figure 43, for example, and travel vertically to, say, the

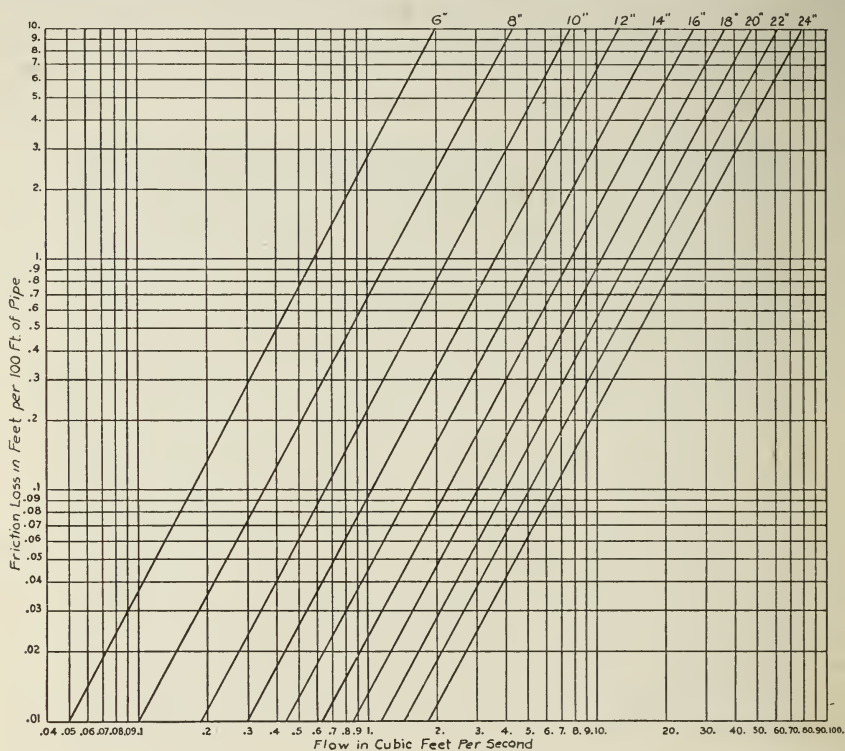


Fig. 43.—Flow of water in steel pipe, based on Williams and Hazen's $C = 100$. The pipe size given is nominal inside diameter. (See reading-reference 8.)

12-inch pipe size. The friction loss or head loss or hydraulic gradient necessary for the steel pipe is 0.7 foot per 100 feet or 0.007 foot per foot; and on figure 44, following the same procedure, the head loss in the dry-mix 12-inch concrete pipe for 3 c.f.s. flow is 0.68 foot per 100 feet = 0.0068 foot per foot.

The fact that a pipe line can disregard grade changes within reasonable limits is very important too. If an open ditch is run down into a swale, there is no way to bring the water up out of that swale again except by pumping. A pipe being closed does not lose the potential elevation by descent into the swale, but uses the water pressure on the high side to force the water up and out of the low side of the swale.

Air trapped in the water tends to collect at high points on a pipe line; one must put small-diameter vents at such points, raising them high enough above

the surface of the ground so that they will not overflow. Without the vents, the collected air acts as a blockade, reducing the flow.

Figure 45 illustrates the conditions along a typical pipe line of uniform size throughout. The pump at the left discharges into a box, A; and the pipe proper takes off near the bottom of that box. The pipe can be assumed to run parallel with the ground surface at a distance 30 inches below the surface. It climbs

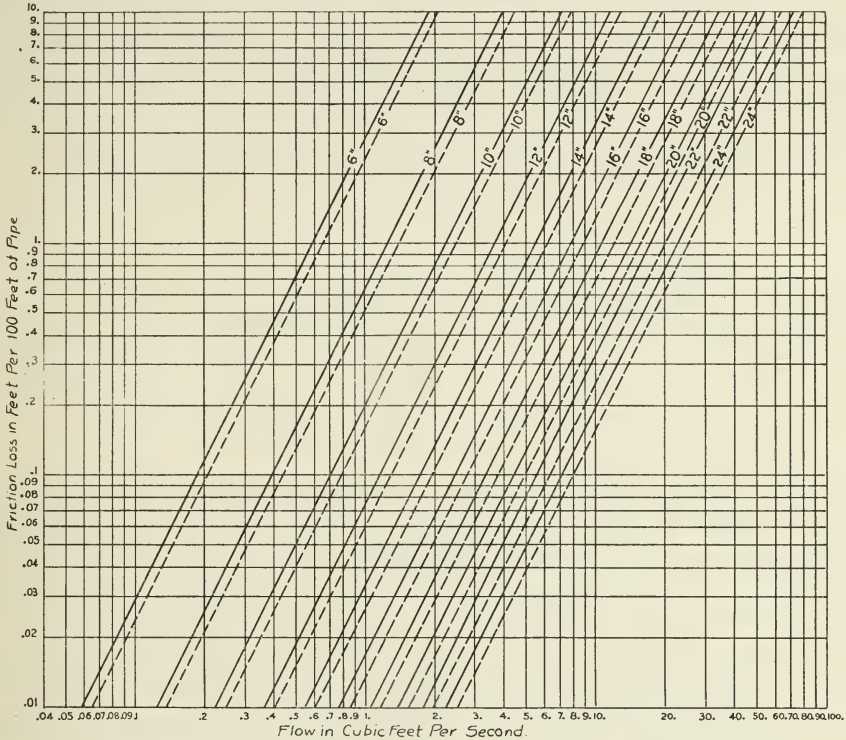


Fig. 44.—Flow of water in concrete pipe, based on F. C. Scobey's data covering dry-mix (heavy line) and wet-mix (dashed line) pipes. (See reading reference 11.)

from A to B, where there is a ridge crest, then drops toward C and D. At C is located another box, or stand, on the line; and at D is a swale with the right-hand side higher than the left hand. From the high side of the swale the pipe again descends to the box, E, where the flow is being discharged. When the pump is running, water stands in boxes A, C, and E; and a straight line will pass through these surfaces, which would correspond to the surface of the water in an open ditch of uniform cross section and construction, handling the pump discharge between the pump and E. This line is also the hydraulic gradient of that size pipe line for the flow carried. Obviously, a ditch constructed on this same line across the piece of land shown would encounter many difficulties. It must be built up very high at A and across D and again presumably at E. The pipe has the same hydraulic gradient a ditch could use, but disregards the

changing grade of the ground. This grade or hydraulic gradient depends on the friction coefficient of the pipe. The height of the hydraulic grade line above the pipe at any point is the static water pressure in the line at that point in feet of water (2.31 feet of water = 1 pound per square inch). If the pipe is rough, the slope from water surface at *A* to water surface at *E* will be larger than if the pipe is smooth. The ditch behaves likewise. If the pipe distance from *A* to *E* is 1,500 feet and the elevation difference between the two water surfaces is 3 feet, the hydraulic gradient is 2 feet per 1,000 feet = 0.2 foot per 100 feet; and the flow passing is the maximum which will pass through the pipe with that loss in head or with that hydraulic gradient. If the pump discharge is increased and the water level at *E* stays constant, the pipe must carry more water, and this will involve more friction. A steeper gradient will then be necessary. To provide it the water at *A* must rise higher in the box. The water at *C* also rises

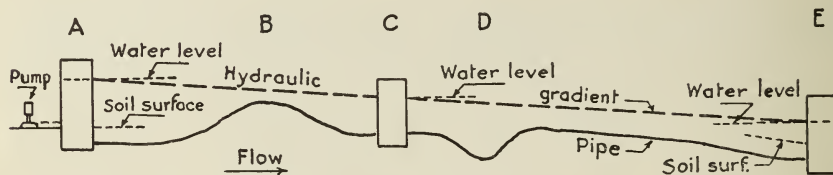


Fig. 45.—Typical pipe-line installation.

to keep the line between water surfaces at *A* and *E* straight. The new gradient might have a slope of 3 feet per 1,000 feet = 0.3 foot per 100 feet, and the flow through the pipe would be the maximum for this gradient. Another use of a figure plotting the hydraulic gradient is to be sure the pipe line will run full. Below the hydraulic grade line there is water pressure in the pipe equal to the vertical distance from the pipe to the line. If the top of the pipe goes above this hydraulic gradient, it goes above the line of zero pressure, and the pipe runs partly full or not at all, unless a sufficient partial vacuum exists along that section of the line. Trouble will result; the condition should be avoided. In addition, any stand along a pipe line must be higher than the hydraulic gradient or it will overflow. Because the designer can draw the hydraulic gradient before constructing the line, he can avoid locating any section of his pipe above that critical line and can dimension all stands with safety. Figures 43 and 44 illustrate the friction-loss (feet loss per 100 feet of line) capacity characteristics of steel and concrete. These curves, being based on average pipe conditions, are safe for general application to farm problems. As in ditch or canal jobs, where a considerable installation is proposed, more accurate and economical design might be secured from more specialized data (1, 11).

Some typical problems will illustrate the use of these charts (figs. 43 and 44) and may help to clarify the discussion of pipe-line behavior.

Problem 1.—What size concrete pipe line will be required to carry 1,500 g.p.m. between points *A* and *B* (fig. 46) located 1,540 feet apart? Elevation of ground surface at *A* = 56.00 feet; at *B*, 64.46 feet. Water stands 10 feet

above the ground surface at standpipe at *A*, and water discharges at level of ground surface at point *B*. Water-surface elevation at *A* = $56 + 10 = 66$ feet, and at *B* = 64.46 feet, or a difference in elevation of 1.54 feet. The length of pipe line = 1,540 feet, and the drop in 1,540 feet = 1.54 feet. The gradient = $1.54 \div 1,540 = 1 \text{ foot} \div 1,000 = 0.1$ foot per 100 feet.

This problem resolves itself to the question of what size line is required to handle 1,500 g.p.m. when the available friction loss can be 0.1 foot per 100 feet = hydraulic gradient. Referring to figure 44, the left-hand side gives friction loss per 100 feet of line. Then follow horizontally on the 0.1 foot per 100-foot line to the right till the vertical line from 1,500 g.p.m. (3.33 c.f.s.) is inter-

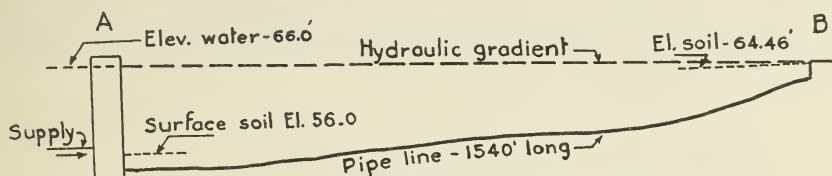


Fig. 46.—Detail of pipe line for piping problem 1.

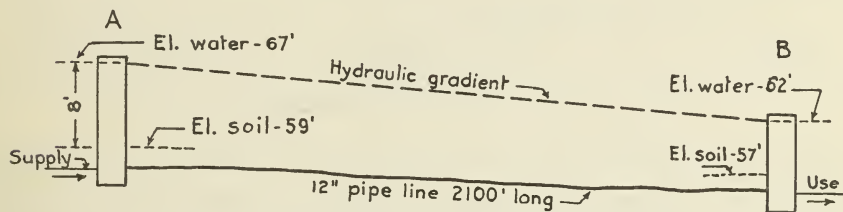


Fig. 47.—Detail of pipe line for piping problem 2.

sected (see fig. 3). The line for the 18-inch pipe passes diagonally through the intersection. An 18-inch pipe will be required. This problem illustrates several interesting points. First, the hydraulic gradient, which is the slope of pressure loss along the pipe, disregards soil-surface levels and concerns itself only with free-water surface levels. Second, the standpipe at *A* must be built somewhat higher than 10 feet above the ground surface there, or it would be subject to frequent overflows. Incidentally, a standpipe 10 feet or higher is not always satisfactory if valves must be operated from the top by means of a ladder. In the present case, the land required the excess height on this standpipe to permit the desired flow through an 18-inch pipe.

Problem 2.—It has become desirable to increase the flow through a 12-inch concrete line (fig. 47) as much as possible by raising a certain low stand at *A* to a clear working height of 8 feet above the surface of the ground. The next stand at *B*, downstream, will spill over its top 5.2 feet above the ground surface if too much is passed beyond it; this is assuming that *B* is the limiting stand, incapable of being raised for some reason. The height of ground at *A* = 59 feet; at *B*, 57 feet. The distance between *A* and *B* = 2,100 feet. Assume a maximum safe rise of water at box *B* = 5 feet above the soil surface. (This leaves only

0.2 foot freeboard; except in this extreme case, this is less than should be allowed.) Here is the solution:

Elevation water (maximum) at A = 67 feet

Elevation water 0.2 foot under top stand B = 62 feet

Elevation difference, A to B, = 5 feet

Hydraulic gradient = $\frac{5}{2,100} = 2.38$ feet per 1,000 feet = 0.238 foot per 100 feet.

From figure 44, left-hand side, giving friction loss per 100 feet of line, follow value 0.238 foot per 100 feet to right to intersection with 12-inch pipe size diagonal line; then follow vertically from this intersection to the bottom line of the figure where capacity of the line is given in cubic feet per second. Under these conditions, a 12-inch line will handle 1.78 or 1.8 c.f.s., approximately, or 800 g.p.m. (from fig. 3).

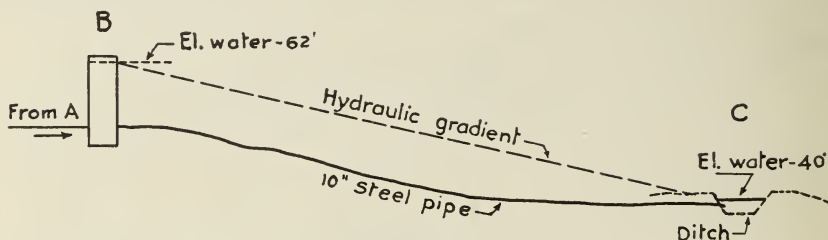


Fig. 48.—Detail of pipe line for piping problem 3.

In this problem we note that, on a long pipe line, the flow that can be accommodated in any section depends often on the flow through some other section of the pipe. In other words, if something downstream from B in this problem did not cause the water to back up at B, there would be a steeper gradient from A to B. This fact is illustrated in problem 3, which follows.

Problem 3.—All the water entering B from A leaves B through a 10-inch steel pipe discharging into a ditch below its water surface, which is at elevation 40 (see fig. 48, an extension of fig. 47). How long is the 10-inch pipe?

The flow into B = 1.8 c.f.s. This same flow must go on or spill over the top of the stand at B; and so 0.2 foot freeboard was kept in the hope of preventing this spill. The flow from B to the ditch = 1.8 c.f.s. The gross drop equals elevation of water surface at B (62 feet) less water-surface elevation in ditch (40 feet), or 22 feet. Using figure 43 for flow through steel pipe and starting from the bottom line of capacity at the point corresponding to 1.78 c.f.s., follow a vertical to the 10-inch pipe size diagonal line; then follow a horizontal line to the left side of the figure to read the corresponding friction loss in feet per 100-feet of pipe. The necessary loss to pass this flow through the 10-inch pipe is 0.66 foot per 100 feet. The loss necessary per 100 feet of line = 0.66 foot equals the hydraulic gradient, and the gross loss in this 10-inch line = 22 feet.

The line, therefore, must be $\frac{22}{0.66}$ hundred feet long to accumulate this loss, or about 3,300 feet of 10-inch line.

Problem 4.—How high will the water stand at point *A* (see fig. 49) if the 14-inch concrete line leading from *A* must pass over a ridge before descending on the other side under the following conditions? Elevation top of inside of pipe at *A* = 57 feet; elevation top of inside of pipe at *B* (top of ridge) = 62 feet; pipe distance *A* to *B* = 1,350 feet; and flow from *A* to *B* = 840 g.p.m. = 1.87 c.f.s. (from fig. 3). In addition, there is an air vent at *B* which would prevent the line from acting as a siphon, and would prevent overflowing.

Though this problem reverses the previous ones, the reasoning remains similar. If the pipe is full to the top at *B* and is standing at *A* to keep the section *A* to *B* filled, then the level of the water at *A* and that at the top of the pipe at *B* will be the same, or 62 feet. The hydraulic gradient is zero, there being no flow. As soon as water starts to move from *A* to and past *B*, friction results, and a hydraulic gradient is necessary. If 1.87 c.f.s. is to pass from *A*

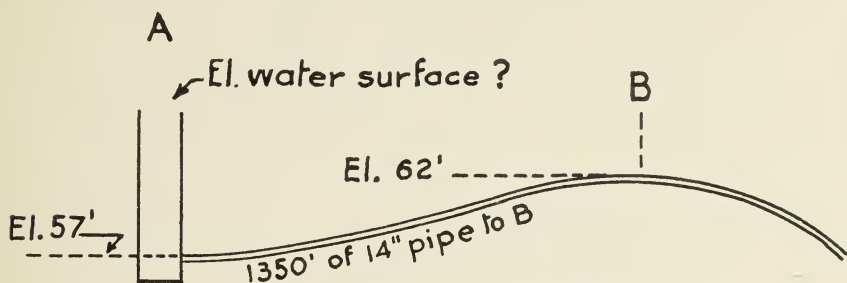


Fig. 49.—Detail of pipe line for piping problem 4.

to *B*, the elevation of the water at *A* must be built up above the level of standing water (elevation, 62 feet) far enough to give the required hydraulic gradient. Using figure 44 for concrete pipe for the flow 1.87 c.f.s., on the bottom line of the figure, move vertically to the solid diagonal 14-inch pipe line; and from that intersection proceed on a horizontal line to the left-hand side of the figure. Here the friction loss per 100 feet of line is read as about 0.12 foot. Since the distance from *A* to *B* is 1,350 feet, the total friction for that distance

would be $\frac{0.12}{100} \times 1,350 = 0.0012 \times 1,350 = 1.62$ feet. The water must rise

about 1.6 feet above the 62-foot elevation in stand *A* to put the required amount through the 14-inch line to *B*. If the line from *B* onward to the right is also 14 inches and slopes at the rate of 0.12 foot per 100 feet or faster, this flow will pass along it freely, with the grade meeting or exceeding the needed hydraulic gradient. In other words, under those conditions the line from *B* onward descends as fast as the flow requires, or faster. If, on the contrary, the grade from *B* on is less than the friction loss resulting from passage of this volume of water, the flow will cease to pass on freely by *B* and will react to the resistance caused by friction. This reaction will be observed at *A*, the first open stand behind the resistance. If the needed additional gradient totals 5 feet from *B* on to the point of escape, then the water in the stand at *A* will

rise 5 feet more, supplying the necessary head for the steeper gradient. If stand *A* is not high enough to permit an additional 5-foot rise of water in it, then it will spill the excess it cannot deliver; and the line will settle down to the passage of what the available hydraulic gradient at the elevation of spill permits—a smaller volume than the 1.87 c.f.s. it can handle from *A* to *B*. A pipe with occasional vents on it is like a chain, no stronger than its weakest link: the line as a whole can handle no more flow than the poorest section will carry. For this reason, design and not chance governs the accurate selection of pipe sizes, lengths of run, and gradients to accomplish the construction of a balanced distribution system. Since a closed pipe line without open standpipes acts as a unit, excess grade at one section can be made available at some point below. As the net result, such a line behaves like any short line between open stands; that is, the true hydraulic gradient from inlet- to outlet-water surfaces governs the flow. The pipe should not pass over that gradient at any point; otherwise the flow will suffer because the pipe tends to run partly full.

A buried pipe line is difficult to clean. If possible, the water it carries should not contain solid matter (sand, silt, vegetation). This foreign material reduces the net cross-section area in the pipe at the point of a concentration. Because of this reduction in area, the velocity must be greater to accommodate the same flow; and the result is an increased friction loss. A system that functioned perfectly before trash accumulated may later become unoperable. Once fouled, the pipe line can be returned to its original operation performance only when the solids have been sluiced out. This cleaning process may be difficult and expensive; precautions to eliminate foreign material in advance may save much expense at a later date. Incidentally, certain chemical fertilizers injure concrete pipe if transmitted through it to the fields; they should be handled with care (14).

Control Devices on Pipe Lines.—The control devices used on pipe lines have two major purposes: protection of the line itself, and effective delivery of the water. Certain devices satisfy both these requirements; others have only a single usefulness. Standard concrete irrigation pipe is not reinforced, so it cannot be used where high internal pressure is possible. Its weakness comes chiefly from its lack of reinforcement. To protect the usual concrete pipe systems, open standpipes are commonly used. When these are used, no water pressure greater than the spill height of these stands can be reached. In addition, air trapped in the line can escape at such stands. Trapped air can dam up a pipe if it cannot escape, as on top of the ridge illustrated at *B* in figure 49 of problem 4. If a standpipe were located at *B* in this figure, or if one of the small commercial air-release valves as suggested (see fig. 50) were put there, the air could escape, thus eliminating the hazard, and also possibly preventing siphoning—a feature which might be favorable at that point on occasions. A pipe like the one in figure 41 might run partly full at *B* if air were trapped as a large bubble at the highest point of the pipe on the crest of the ridge. The possible flow in the pipe line would then be cut down; and, if the 1.87 c.f.s.

were supplied at stand A, the design would seem to fail. Bleeding of the trapped air would eliminate this block and enable the design conditions to be met.

If a pump discharges directly into a concrete line without any air relief nearby, the advancing stream may encounter an air pocket of the type just

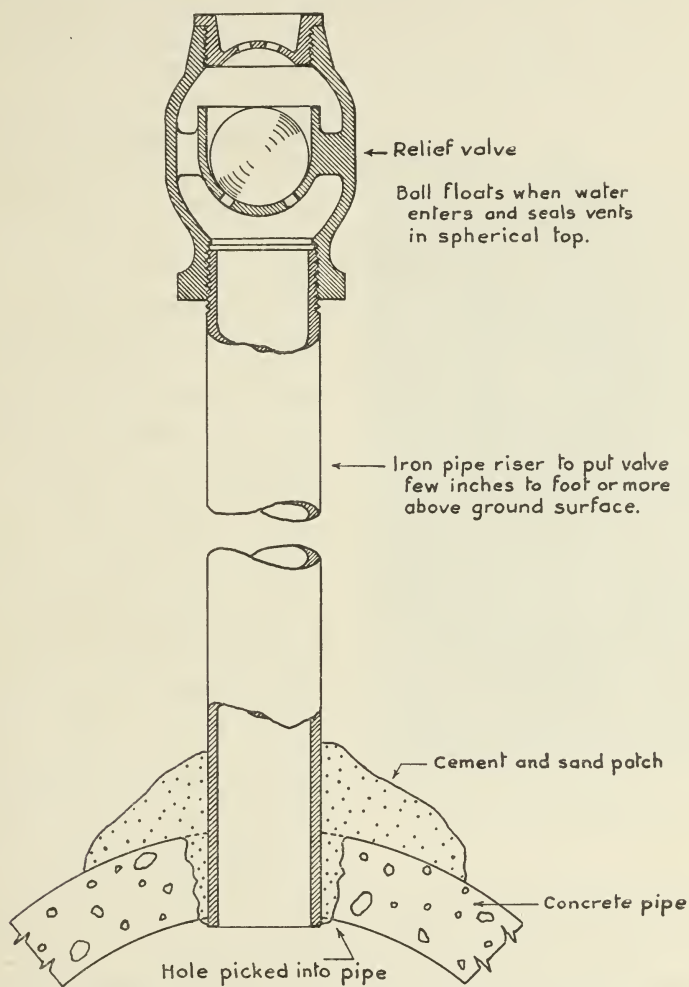


Fig. 50.—Small ball float-relief valve for venting air on pipe lines.

described and, being unable to pass quickly, may cause a sudden and severe shock. A sharp bend in the pipe can effect a similar shock condition. Because, with this "water hammer," the stresses are many times greater than normal, and pipe or joints often fail when a pump is connected directly to a concrete line. A high standpipe at the point of connection with the pump will help to eliminate water hammer. As an additional precaution, some sort of slip-joint connection may be inserted in the discharge pipe between the pump and the

standpipe. Otherwise expansion and contraction movements may cause broken standpipe, or shifting of the pump, or both. Where the pump lifts considerable amounts of sand, the safest practice is to pass the pump discharge through a large, closed concrete or metal box where the velocity will be much reduced and the sand will settle out before entering the pipe line. Such a box is called a sand trap. Without a sand trap, under these conditions, the pipe system may become clogged and, as a result, useless.

Small-diameter stands (24 inches or less) have value as air-relief points and limited value as surge chambers. Small-diameter, light-weight pipe (2 to 4

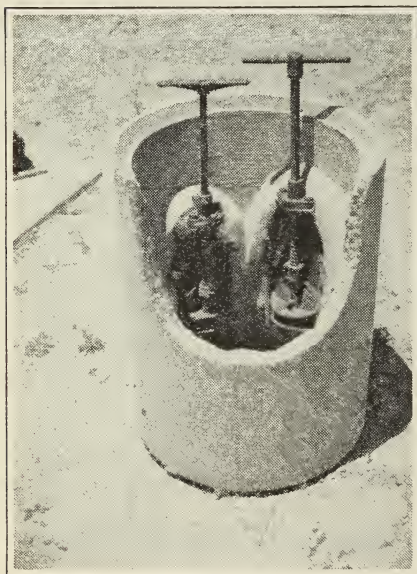


Fig. 51.—Cut-away view of two small gate valves installed in a section of concrete pipe to be used as a standpipe.

inches) can be used as air vents only. Larger stands (over 24-inch diameter or rectangular sections) will serve several purposes. They relieve trapped air and, being larger, absorb surges more easily. If a block occurs down stream, the water rises more slowly in them, and the necessary hydraulic gradient is arrived at gradually. In addition, larger standpipes provide sufficient diameter to admit several lead-off or take-off pipes if needed. When several take-off pipes are installed at a stand, valves are desirable at the inlet of each; they are generally located inside the standpipe (fig. 51), with valves at the stand for every take-off line. All lines can be used, if need be, while one is being worked upon. Also, the operator need not fill all other take-off lines when using water through any one of them. Large standpipes thus equipped with valves

and with connected lateral pipes become control points, division points, or diversion boxes, on the system, regulating the delivery. It may be desired to carry water down two or three laterals from such a regulating stand. One of these laterals may be short or sharply sloped, whereas the others are long and lack a good grade. If the short or sharply sloped one had no gate, it might take almost all the supply, the other two getting only a fraction of the flow. With a gate before this free-flowing outlet, and on the other two as well, the operator can restrict the flow to the ones having easiest get-away conditions and can balance the flow to all, or arrange any desired distribution between the outlets in use and still keep the stand from spilling over. Figure 51 also shows typical gates for use at stands. A special adaptation of the open stand is the closed stand or closed division box, which, as the name implies, is a roofed intersection point for several pipe lines. The control valve stems are run through

stuffing boxes in the roof (fig. 52). Their chief application is at points where division boxes are a necessity; but because of the hydraulic gradient conditions, water would back up too high in an open stand to permit easy operation of the valves (over 10 feet above the ground surface). Such a closed stand might have a small (4 to 6 inches) sheet-iron stack at its roof center to vent air and so might be partially effective in absorbing surges. Such stacks can be guyed and may stand 5 to 10 feet or higher than normal water levels at the stand. Figure 52 illustrates a closed stand with stack and control valve.

Another special stand (fig. 53) is the divided type, in which a vertical wall across the middle makes the structure a two-compartment unit. The water supply enters on one side of such a stand, and one or more pipes take off from each compartment. For greatest flexibility all take-off pipes have valves; and a valve is also put near the bottom of the dividing wall over a port joining the two sides of the wall. Regardless of variable use by the take-offs on the other side of the wall, those on the supply side can be assured of a constant height of water in the stand as long as the dividing wall-gate valve is closed. Such a steady height of water assures steady flow in those supply-side take-offs, and, if they supply ditches, there is little danger of breaks in the ditches under these conditions. Without the wall, variable use by other lines would create variable supply to the ditch lines and probably frequent over-supply, with broken ditch banks resulting. These two special stands—the closed and the divided—are chiefly delivery structures.

Concrete pipe lines carrying water down steep grades, as in hilly orchard localities, must have divided stands at frequent intervals (fig. 54), or a substitute for these stands available in the form of a float-operated valve which gives complete control of the water. These stands or the float valve actually break the pipe into short lengths so that each unit can have but 3, 6, or 8 feet of water pressure on it. Water can be drawn off from the upstream disk-type valve (fig. 54, detail b). Water can be used at both top and bottom of such a line at any time or at any intermediate point. If the divided overflow stands were not located on such a steeply sloped line, the pressure at the bottom of the line would be excessive. Water, when introduced, would plunge to the bottom and must stop at the end. In all probability, the resultant shock would

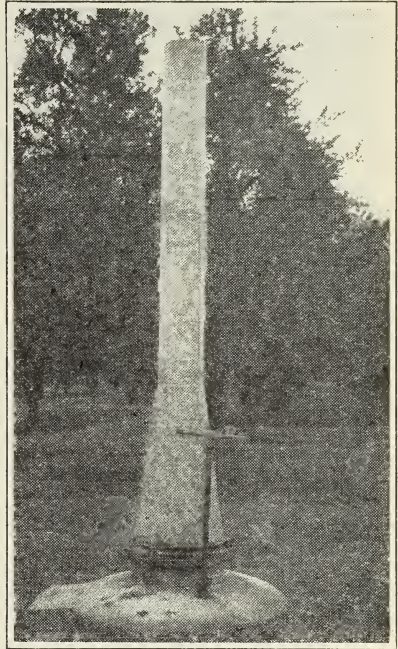


Fig. 52.—Metal stack or riser on low concrete stand. Stuffing box of valve stem shown in front of stack.

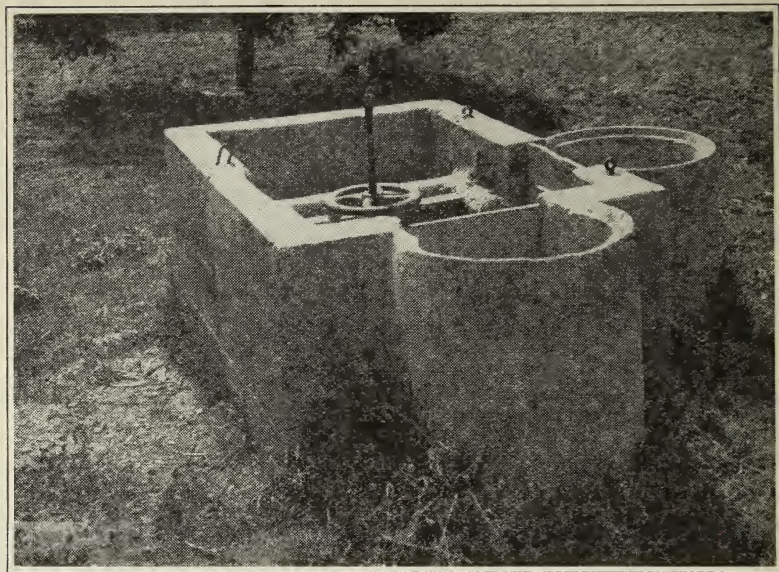


Fig. 53.—Special stand or box with dividing walls. Note use of pipe sections adjoining rectangular field-cast main box.

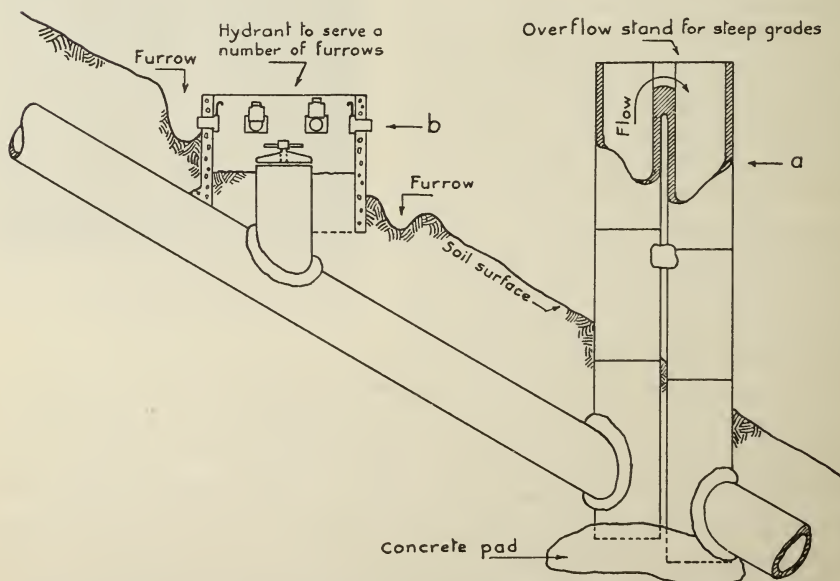


Fig. 54.—Overflow stand (a) and pot hydrant (b) for hillside pipe-line system.

burst the pipe. If it did not, the pressure resulting from the water column would make it spurt out of any open valve located near the bottom of the line. If water was being taken out at the bottom of continuous pipe, the hydraulic gradient in the pipe might drop below the surface of the soil near

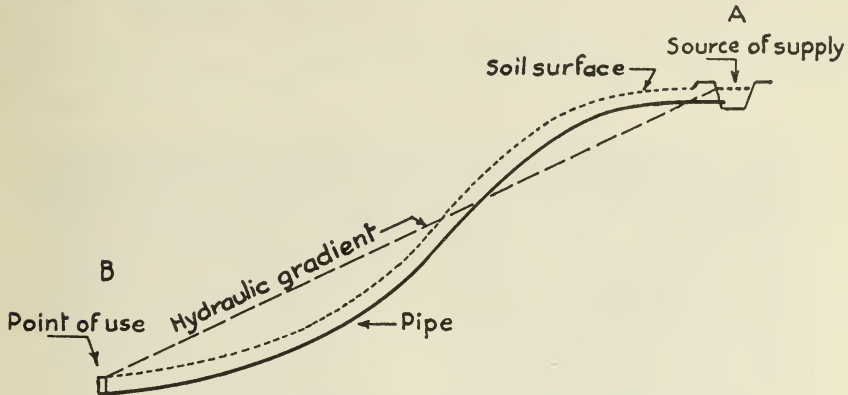


Fig. 55.—Hydraulic gradient resulting from use of water at bottom of steeply sloped pipe system.

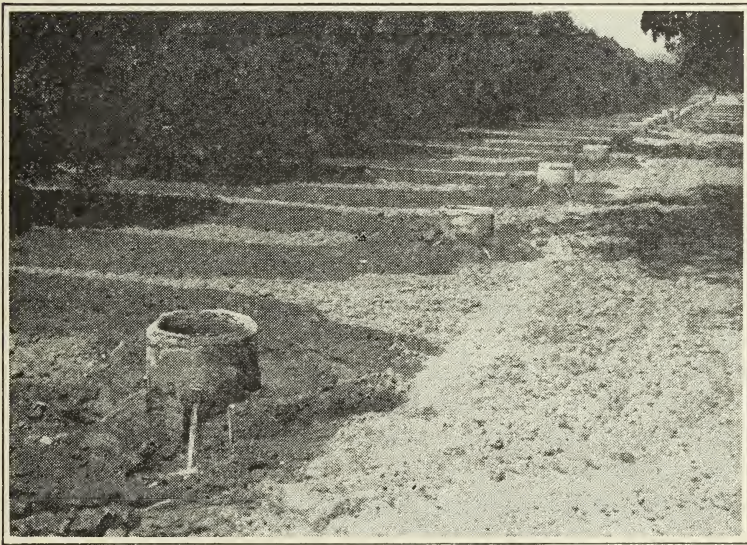


Fig. 56.—Pot hydrants supplying furrows on flat land.

the upper end and water could not be used at that point under those conditions (fig. 55). This recalls the requirement, previously noted, that the hydraulic grade line must lie above the soil surface or water will not flow onto the soil from a pipe.

Detail *b* of figure 54 (see also figure 56) illustrates a multiple outlet or hydrant used on hillside irrigation of orchards. These structures are equally

useful on flatter land for irrigating row crops or furrows (fig. 56). They simulate the furrow tubes used in ditches and are, in fact, identical in their use. The group of short furrow gates is congregated around the short section of pipe called a pot, and set over the valve that covers the connection to the supply

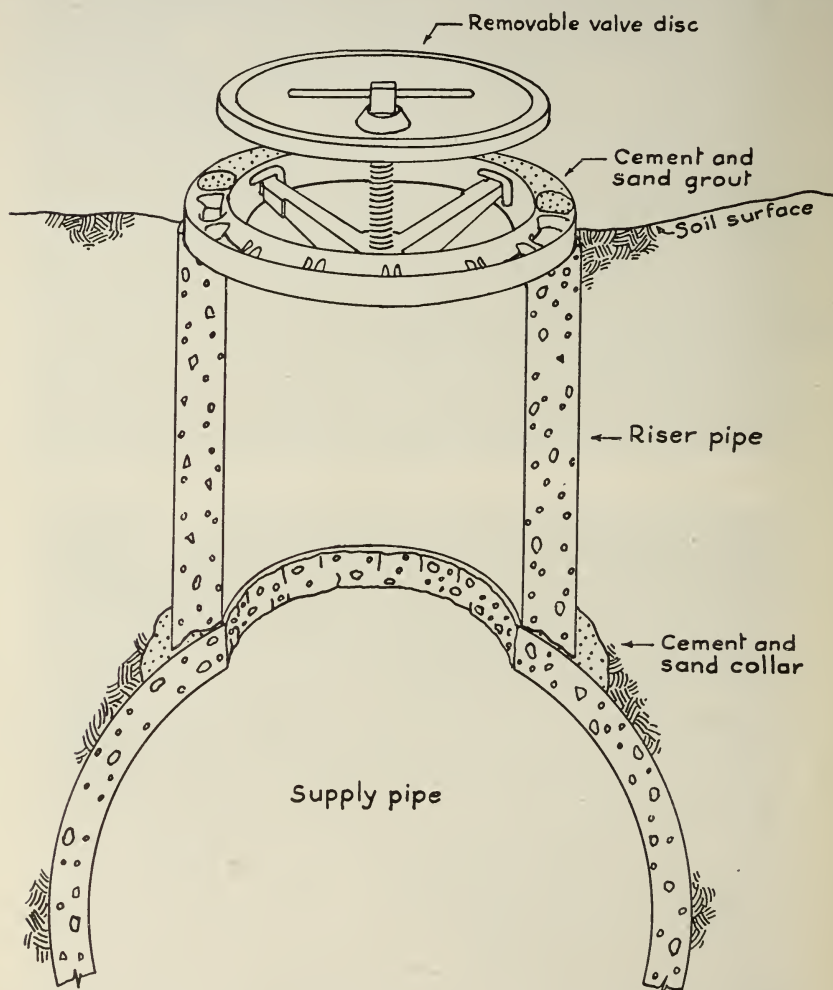


Fig. 57.—Typical riser and "alfalfa" valve for supplying a ditch or for flooding.

pipe; the furrows are curved about the structure to catch the spill from the individual tubes. Sometimes distributing arms made of 2-inch or larger light-weight pipe are attached to the pots and distribution to the furrows is made through holes in the arms. The flow to furrows can be controlled by manipulation of the main valve (usually called an orchard valve) or by use of the slide gates on the separate tubes or by both means. The main control valve must be regulated so that the pot is not overflowed.

Where irrigation is done by flooding as in orchards and on alfalfa and other

crops, the concrete retaining wall and furrow tubes are not needed and a simple disk valve (usually called an alfalfa valve) flush with, or a few inches below, the surface of the ground, is all that is necessary. It is placed on a short connecting pipe cemented to the supply pipe (fig. 57). Such outlets can also be the supply source for a ditch or for several ditches (fig. 58). Whatever the application, the flow from the mouth of the opened gate must not be permitted to erode the soil. Often vegetation is grown about the gate to act as a buffer against such erosion.

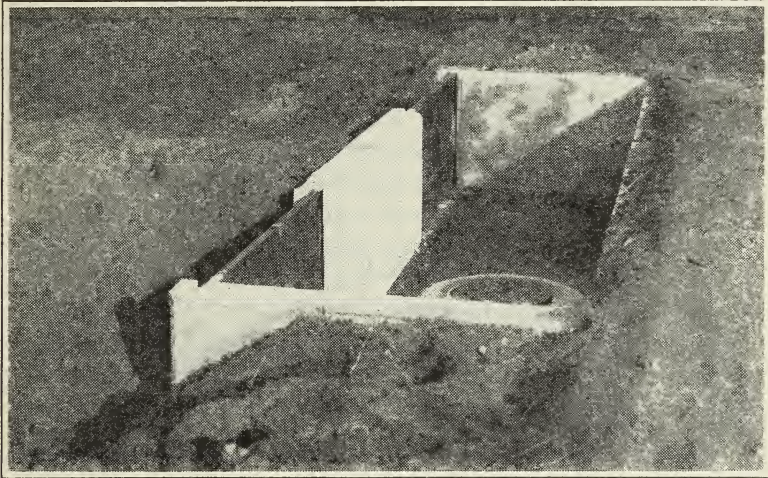


Fig. 58.—Special installation of disk valve on riser to feed one or more ditches or to irrigate a pair of adjoining fields. The enclosure prevents erosion.

As with the ditch structures, there are many types of pipe structures. The basic ones have already been discussed, with certain possible variations. Presumably, using the description supplied, the irrigator can make any further variations necessary for his system.

The remaining structures are also delivery devices with particular application to the release of water reasonably near the area of use. They comprise the risers and accompanying valves or gates used for supplying ditches, fields, or furrows. The risers (figs. 57 and 58), as their names imply, are the vertical ducts that come to the soil surface from the buried pipe line. The tops of these ducts, which are usually sections of concrete pipe cemented to a hole on top of the supply line, must have valves in them so that water can be released only at the point or points desired. Since the use of steel lines for irrigation distribution is uncommon, no special equipment has been produced to go with such lines for irrigation service only. In general steel is not so long-lived as concrete.

Sprinkling Systems.—Sprinklers and sprinkling systems comprise a distinct development in irrigation, and a thorough discussion of them is available (12).

Inverted Siphons.—Inverted siphons are relatively short pipe lines, usually installed to conduct the flow of a ditch under some surface obstruction such as

a road or railway embankment. One of these applications has been discussed in connection with passageways across field ditches. A typical design problem for such a siphon is solved as follows: suppose that the length of the concrete pipe siphon is 54 feet from inlet to outlet; that the ditch handles 2 c.f.s.; that the water must not stand in the ditch at the entrance to the siphon above an elevation of 267.0 feet; and that the water flows in the ditch at the downstream side of the siphon at an elevation of 266.7 feet. The available difference in head = 267.0 feet - 266.7 feet = 0.3 foot, which can be used as friction loss.

The length of siphon, 54 feet, $= \frac{54}{100} = 0.54$ foot. The available loss per

100 feet of pipe $= \frac{0.3}{0.54} = 0.556$ foot per 100 feet of pipe = H . Referring

to figure 44, from $H = 0.556$ on the left side of the chart, move to the right to the vertical line from 2 c.f.s. on the bottom of the chart. Since these two lines intersect between the curves for the 10- and 12-inch pipe sizes (although nearer the 10-inch), the 12-inch pipe size would be installed.

This problem is solved as though an open standpipe were located at each end of the siphon. The elevation of the water surfaces at the inlet and outlet of the siphon will control the flow for any given size of pipe, just as do the elevations of water surface in the stands. Because a siphon is short, as a rule, one can afford to use larger pipe than the design calculations indicate is necessary. Such procedure is wise because of the losses in head at the entrance and exit of the normal siphon. These losses are caused by sudden change in shape from the ditch section to the siphon form; the water does not adjust itself to the change without sacrificing energy in the form of head drop. It is to reduce these losses that throats are provided at the entrance and exit of siphons, as discussed earlier. Even with the throat, one may well use a somewhat larger pipe than the calculations apparently require.

ACKNOWLEDGMENT

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